CITY OF SHOW LOW

PHASE 3
WASTEWATER MASTER PLAN
ALTERNATIVES ANALYSIS

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WP# 113701

Prepared for:

City of Show Low
550 North 9th Place
Show Low, AZ 85901
(628) 532-4040

Prepared by:

Wood, Patel & Associates, Inc.
2051 West Northern Avenue
Suite 100
Phoenix, AZ 85021
(602) 335-8500

Registered Professional Engineer (Sanitary)
CERTIFICATE NO.
12590
K. L.
KNICKERBOCKER
Date Signed 7-25-12
ARIZONA, U.S.A.
Expires: 6/30/2015

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1.0 INTRODUCTION

This document is identified as Phase 3 of the Show Low, Arizona Wastewater Treatment Plant (WWTP) Master Plan (WMP). Reports were previously prepared for phases 1 and 2, which are dated May 2007 and October 2009, respectively. Information in the Phase 1 and 2 reports supplements the information presented in this Phase 3 report. Key elements of this Phase 3 report are as follows:

- Evaluation of alternatives for a new WWTP to treat wastewater for a 20-year planning period
- Evaluation of liquid and solids treatment alternatives
- Evaluation of disposal alternatives of wetlands discharge, reclamation/reuse, irrigation and stream discharge
- Identification of regulatory approvals and permits required

2.0 CURRENT CONDITIONS

The City of Show Low wastewater system has been in operation since 1958 and serves residents and businesses within the Show Low City limits and service area. The WWTP was upgraded in 1985 to treat an average flow rate of 1.42 million gallons per day (MGD) and a peak hydraulic design flow of 4.26 MGD. This was based upon an estimated unit flow contribution of 105 gallons per capita per day (gpcd). The design flow was projected to occur in 2006.

The WWTP has not reached this design flow capacity. The current average annual flow to the WWTP is approximately 1.02 MGD, the summer peak flow is approximately 1.25 MGD and the Maximum Month Daily Flow was 1.57 MGD.

The City is authorized to discharge treated wastewater in accordance with the facility's AZPDES permit

2.1 Permits

The City of Show Low operates its wastewater treatment plant under two permits issued by the Arizona Department of Environmental Quality (ADEQ). The Aquifer Protection Permit (APP) is dated October 28, 2002 and authorizes the city to operate a wastewater treatment plant, treating a maximum wastewater flow of 2.46 MGD. The permit is valid for the life of the facility.

The second permit, Arizona Pollutant Discharge Elimination System (AZPDES), authorizes the City to discharge treated wastewater to the wetlands. The permit is dated September 25, 2006 and was due to expire on September 25, 2009. The City has submitted an application to renew the permit, and ADEQ has prepared a draft permit (dated December 28, 2011), which the City has responded to. The final permit has not been issued and the City continues to discharge from the wastewater treatment facility in accordance with the terms of the 2006 AZPDES permit modified as described below. We anticipate the final report by the end of 2012.

2.1.1 Nitrogen

The City of Show Low has petitioned ADEQ for a variance of the proposed NH_3-N limits for the WWTP. The variance application was submitted to ADEQ on May 15, 2009, and ADEQ responded on December 28, 2011 as part of the draft AZPDES permit requiring compliance with the standard within three years of the date of the final permit. Submittals including a sampling plan for ammonia and other reports are required and detailed in the compliance schedule. Within thirty-six (36) months after the effective date of the permit the facility is required to be in compliance with the water quality standards for ammonia as per R18-11-104.C (refer to Appendix A – ADEQ Nitrogen Limits).

The Phase 2 report discussed nitrogen removal characteristics of the existing and expanded lagoons and polishing wetlands. Calculations for the nitrification process indicate that complete nitrification will not occur within the existing lagoon treatment process due to colder temperatures in the winter months (where water temperatures are at or below 5° C (41° F)), and insufficient mixing, regardless of how much air is provided. The report recommended improvements to the existing lagoons required to meet the new ammonia standard through 2015 (summer flow of 1.71 MGD), using the previously projected loadings. Subsequently, the City launched a maintenance program that significantly reduced the loadings and the improvements were revised and are presented in Section 4 of this report.

2.1.2 Selenium

Selenium concentrations in the effluent from the polishing ponds have exceeded the allowable permit concentrations. The City of Show Low petitioned ADEQ for a variance for selenium. The variance application was submitted to ADEQ on May 15, 2009, and ADEQ responded on December 28, 2011 as part of the draft AZPDES permit granting a variance for selenium in order to allow time by the end of the first three years of the permit term to propose and conduct a study for the development of a site-specific standard for selenium or select an alternative method to achieve compliance with the applicable water quality standard for selenium. The City must conduct a study for the development of a site-specific standard for selenium or determine an alternative method to achieve compliance with the applicable water quality standards for selenium. If the City decides to conduct a study for the development of a site-specific standard for selenium, the U.S. Fish and Wildlife will be contacted for information and assistance in the study design, implementation, and/or analysis. Additionally, it is required the study use the "Protocol for Aquatic Hazard Assessment of Selenium" developed by Dennis Lemly (2005).

3.0 POPULATION AND FUTURE GROWTH

This WMP addresses the City's projected wastewater needs for 20 and 40 years into the future. Future flow projections require first estimating the population growth. Several population projections were previously presented in other reports. This report modifies the population projections of the previous reports.

The City of Show Low 2010 population was determined by the U.S. Census Bureau to be 10,660 persons. This was 1,533 less than was previously estimated by the Arizona Department of Economic security (AZDES) for 2010. Consequently the previous population growth projections were reduced by this difference for each year through the year 2031. Table 3-1 shows the population projections through year 2031. It shows that the City of Show Low is projected to have a year 2031 permanent population of 18,374 and a total seasonal population of 25,724 persons.

Table 3-1 - Projected Population and Flows

Year	2010 Census	AZ DES Projection	Δ
2010	10,660	12193	-1,533

	Populations				Design Flows Projections							Estimated Flows		
Year	AZ DES Projection	Adjusted AZ DES Projection[1]	Summer[2] Population	Total Seasonal Population	Avg. Winter Flow[7] (mgd)	Avg. Summer Flow[3,7] (mgd)	AADF[9] (mgd)	Maximum Month[4] (mgd)	PF[5]	Peak flow[6] (mgd)	Instantaneous[8] Peak flow (mgd)	Annual Flow (mgd)	Maximum Month[4] (mgd)	Peak flow (mgd)
2010	12,193	10,660	4,264	14,923	0.91	1.27	1.00	1.47	1.79	1.78	2.62	0.91	1.40	1.63
2011	12,641	11,108	4,443	15,551	0.94	1.32	1.04	1.53	1.78	1.85	2.72	0.95	1.46	1.69
2012	13,086	11,553	4,621	16,174	0.98	1.37	1.08	1.59	1.77	1.92	2.82	0.99	1.52	1.75
2013	13,526	11,993	4,797	16,790	1.02	1.43	1.12	1.65	1.77	1.98	2.92	1.02	1.58	1.81
2014	13,957	12,424	4,970	17,393	1.06	1.48	1.16	1.71	1.76	2.05	3.01	1.06	1.63	1.87
2015	14,380	12,847	5,139	17,986	1.09	1.53	1.20	1.77	1.76	2.11	3.11	1.10	1.69	1.93
2016	14,797	13,264	5,306	18,570	1.13	1.58	1.24	1.82	1.75	2.18	3.20	1.13	1.74	1.99
2017	15,205	13,672	5,469	19,140	1.16	1.63	1.28	1.88	1.75	2.24	3.29	1.17	1.80	2.04
2018	15,603	14,070	5,628	19,698	1.20	1.67	1.32	1.94	1.74	2.30	3.38	1.20	1.85	2.10
2019	15,993	14,460	5,784	20,244	1.23	1.72	1.35	1.99	1.74	2.36	3.46	1.23	1.90	2.15
2020	16,370	14,837	5,935	20,772	1.26	1.77	1.39	2.04	1.74	2.41	3.55	1.27	1.95	2.20
2021	16,739	15,206	6,082	21,288	1.29	1.81	1.42	2.09	1.73	2.47	3.63	1.30	2.00	2.25
2022	17,098	15,565	6,226	21,792	1.32	1.85	1.46	2.14	1.73	2.52	3.70	1.33	2.05	2.30
2023	17,446	15,913	6,365	22,278	1.35	1.89	1.49	2.19	1.73	2.57	3.78	1.36	2.09	2.35
2024	17,782	16,249	6,500	22,749	1.38	1.93	1.52	2.24	1.72	2.62	3.85	1.39	2.14	2.39
2025	18,112	16,579	6,632	23,211	1.41	1.97	1.55	2.28	1.72	2.67	3.93	1.42	2.18	2.44
2026	18,431	16,898	6,759	23,658	1.44	2.01	1.58	2.32	1.72	2.72	4.00	1.44	2.22	2.48
2027	18,741	17,208	6,883	24,091	1.46	2.05	1.61	2.37	1.72	2.76	4.06	1.47	2.26	2.52
2028	19,043	17,510	7,004	24,513	1.49	2.08	1.64	2.41	1.71	2.81	4.13	1.50	2.30	2.56
2029	19,337	17,804	7,121	24,925	1.51	2.12	1.67	2.45	1.71	2.85	4.19	1.52	2.34	2.60
2030	19,625	18,092	7,237	25,329	1.54	2.15	1.69	2.49	1.71	2.89	4.25	1.55	2.38	2.64
2031	19,907	18,374	7,350	25,724	1.56	2.19	1.72	2.53	1.71	2.94	4.32	1.57	2.42	2.68

[1] The AZ DES population projection was adjusted in accordance with the 2010 census population taken in 2010. The difference between the 2010 population and population projection of 1,533 people was subtracted from the projected population through the year 2031.
[2] Seasonal Population increase is approximately 40% based on the City of Show Low Wastewater Facilities Master Plan Report, dated May 2007 by Kennedy Jenks.
[3] The unit flow per capita is 81.2 gpcd for the Design Flow Projections. The unit flow per capita is calculated as the average of the unit flow rates calculated from 2003-2008. The unit flow factor for the Design Flow Projections was calculated based on the AZ DES Adjusted Population. The unit flow factor for the Estimated Average Annual Flow is based on the seasonal population and calculated unit flows for 2008. The 2008 estimated unit flow is 61 gpcd.
[4] Max Month peak factor is based on the 2008 influent wastewater flow data (Jan.08 thru Aug. 08) and is equal to Max Month Flow divided by Average Annual Flow. Max month flow for 2008 is 1.57 MGD in February. The Max Month Flow Factor = 1.57/1.07 = 1.47
[5] Peaking Factor in accordance with A.A.C R18-9-E301.401D
[6] Peak flow is equal to the Average Annual Flow times the calculated peaking factor.
[7] Yearly Flow projections are based on the adjusted AZ DES population projections. Total flow is based on the Total Seasonal population.
[8] Instantaneous flow is equal to the Max Month times the Peaking Factor.
"[9] AADF is the weighted average of the Avg Winter Flow and Avg. Summer Flow where Summer flow is assumed to occur between June and August.
AADF = (273days *Avg Winter Flow + 92days * Avg. Summer Flow)/365days"

4.0 WASTEWATER LOADINGS

4.1 Historic Flow

The WWTP was designed in 1985 to treat an average flow rate of 1.42 MGD, with a peak design flow of 4.26 MGD. The year 2008 average daily flow rate was approximately 1.02 MGD, and the maximum month daily flow rate was 1.57 MGD.

4.2 Projected Flow

Table 3-1 contains flow projections through year 2031. Two categories are identified. These are identified as "Estimated Flows" and "Design Flows Projections".

Estimated Flows

The "Estimated Flows" were determined by multiplying the year 2008 average daily flow (1.02 MGD) by a ratio of the future projected population to the 2008 population. The "Maximum Month" flow projections were established by Annual Flow by the factor 1.57/1.02 or 1.54. This data is NOT the basis for establishing future treatment requirements, but is presented for comparison to the Design Flows Projections.

Design Flows Projections

Due to the seasonal variation in population in Show Low, the wastewater treatment plant design is appropriately based upon a "weighted" value of the winter and summer populations. This information is contained in the "Design Flows Projections" portion of the table. The values in the AADF column are the weighted averages of the Average Winter Flow and Average Summer Flow. The summer flow is assumed to occur between June and August. The formula is expressed as follows: AADF = (273days *Avg. Winter Flow + 92days * Avg. Summer Flow)/365days

The projected Maximum Month flow was determined by multiplying the average annual daily flow (AADF) by a factor of 1.47. The "Estimated Flows" and the "Design Flows Projections" in Table 3-1 are close to the "Design Flow Projections", only slightly higher. Therefore, the projected Design Flows calculated in Table 3-1 may appropriately be used in the Show Low Master Plan. The year 2031 AADF is estimated to be 1.72 MGD and Maximum Month Flow 2.53 MGD.

4.3 Projected Loadings

The Phase 2 report recommended the following constituent concentrations as being representative of the loading characteristics to the Show Low WWTP and should be used as the basis for preparing process calculations to determine the adequacy of the current WWTP process and future WWTP needs.

BOD_5	435 mg/L	TKN	45 mg/L
TSS	435 mg/L	Temperature	72 F Maximum

The Phase 1 Technical Memorandum analyzed the sewage collection system. It concluded that a source of high BOD_5 and total suspended solids (TSS) concentrations was isolated in the old town area, where the sewers slopes tend to be flat and in need of repairs. Low velocity in these sewers, particularly during prolonged dry weather, results in septic conditions and accumulation of solids. This in turn, increases BOD_5 and TSS concentrations of the influent to the WWTP. Subsequently, the City commenced a sewer maintenance plan that included routine flushing of known areas of sewers where accumulation of solids occurred. This maintenance strategy was successful in lowering the BOD_5 and TSS substantially. Graphs of the influent BOD, total Kjeldahl nitrogen (TKN), ammonia, phosphorus, TSS and flow data for 2010 and 2011are presented in Appendix B and illustrate weekly influent variations in these parameters. The following summarizes these data:

Avg. BOD_5	182 mg/L
Avg. TSS	244 mg/L
Avg. TKN	47 mg/L

To quantify the actual peak monthly loading to the WWTP, an analysis of the mass organic and solids loading was performed. Appendix B shows a graph of the mass organic and solids loadings for the WWTP influent from 2010 to 2011.

Influent data for raw wastewater samples obtained at the WWTP headworks was analyzed graphically using a histogram and the frequency of occurrence of values prepared to help highlight the return frequency which can be used to anticipate the extreme and median data. Histograms for BOD, TSS and TKN are presented in Appendix B. Based on these data and a graphical analysis, the following revised values are recommended for BOD, TSS and TKN for sizing treatment processes:

BOD_5	**275 mg/L**
TSS	**300 mg/L**
TKN	**50 mg/L**

4.4 Design Criteria

The City's wastewater is primarily residential in nature. Design period is 40 years, with two phases, each 20 years in length. Table 4-1 identifies the preliminary design criteria for the facility for both phases 1 and 2; however, this Master Plan presents treatment cost estimates for Phase 1 only.

Table 4-1 – Recommended Design Criteria – Show Low WWTP

Parameter	**PHASE 1**[1]	**PHASE 2**[2]
INFLUENT		
Flow, MGD:		
- Peak Month	2.53	3.16
- Peak day	2.94	3.61
Characteristics:		
- Biochemical Oxidation Demand (BOD_5), mg/L	275	275
- Chemical Oxidation Demand (COD), mg/L	550	550
- Total Suspended Solids (TSS), mg/L	300	300
- Total Kjeldahl Nitrogen (TKN), mg/L	50	50
- Ammonia Nitrogen (NH_3-N), mg/L	40	40
- Total Phosphorus (PO4 P), mg/L	9	9
- Alkalinity, mg/L CaCO3	325	325
- pH	6.2 – 9.0	6.2 – 9.0
MISCELLANEOUS		
Influent Wastewater Temperature, °F		
- Winter	40	
- Summer	75	
Ambient Air Temperature, °F		
- Winter – Average Low	26	
- Summer – Average High	87	
Elevation, ft. MSL	6,330	
FINAL EFFLUENT		
Reclaimed Water Class (R18-11-303)	A+	A+
- BOD_5, mg/L	15	15
- TSS, mg/L	15	15
- Total Nitrogen, mg/L	<10	<10
- Total Phosphorus, mg/L	<0.1	<0.1
- Fecal Coliforms, CFU/100 ml	ND	ND
- Turbidity (NTU)	<2.0	<2.0
Reclaimed Water Class (R18-11-305)	B+	B+
- Total Nitrogen, mg/L	<10	<10
- Fecal Coliforms, CFU/100 ml	<200/100 ml	<200/100 ml

[1]Through 2031. See Table 3-1.
[2]Through 2051.

5.0 WASTEWATER TREATMENT ALTERNATIVES ANALYSIS

The following alternatives analysis presents options for future wastewater treatment to meet discharge requirements. The City is currently evaluating obtaining an alternate site to locate a new wastewater treatment facility. The site will be part of a land exchange with the USFS and includes approximately 79 acres located in section 8, T10N, R22E. Alternatives 2b and 3 – 6 are proposed to be located on this site, which is referred to as the USFS site in the remainder of this report. Only Alternatives 1 and 2a are located on the existing WWTP site. The alternatives are generally grouped as follows:

- Alternative 1 - Upgrade the existing lagoons to treat to Class B. This includes modifying the existing lagoons to meet proposed standards.
- Alternatives 2a and 2b – Replace the existing lagoons with an activated sludge Biolac plant at the site of the existing lagoons to treat to Class B+ under Alternative 2a while Alternative 2b evaluates constructing the Biolac facility at the USFS site location.
- Alternatives 3 through 5 – Construct a new facility at the USFS site to treat to either Class B+ or A+. These alternatives include abandoning the existing WWTP.

The analysis of Alternative 1 includes a comprehensive evaluation of the existing WWTP since abandoning the existing facility in favor for a new WWTP will result in application for new permits, the cost of abandonment, a change in operational strategy and significant off-site pipeline routing.

Alternative 2 is further divided into Alternatives 2a and 2b. Alternative 2a includes constructing an activated sludge Biolac plant on the existing treatment plant site. It would take advantage of the existing lagoons and provide B+ treatment levels similar to Alternatives 3 through 5. In addition it avoids the off-site pipeline routing. Alternative 2b evaluates constructing the Biolac facility at the USFS site.

Alternatives 2 through 5 are analyzed as a group because of their similarities while Alternative 1 is analyzed separately. Figure 5-10 shows the approximate location of the USFS site. An estimate of probable construction cost is presented for each alternative as well as estimated operation and maintenance costs.

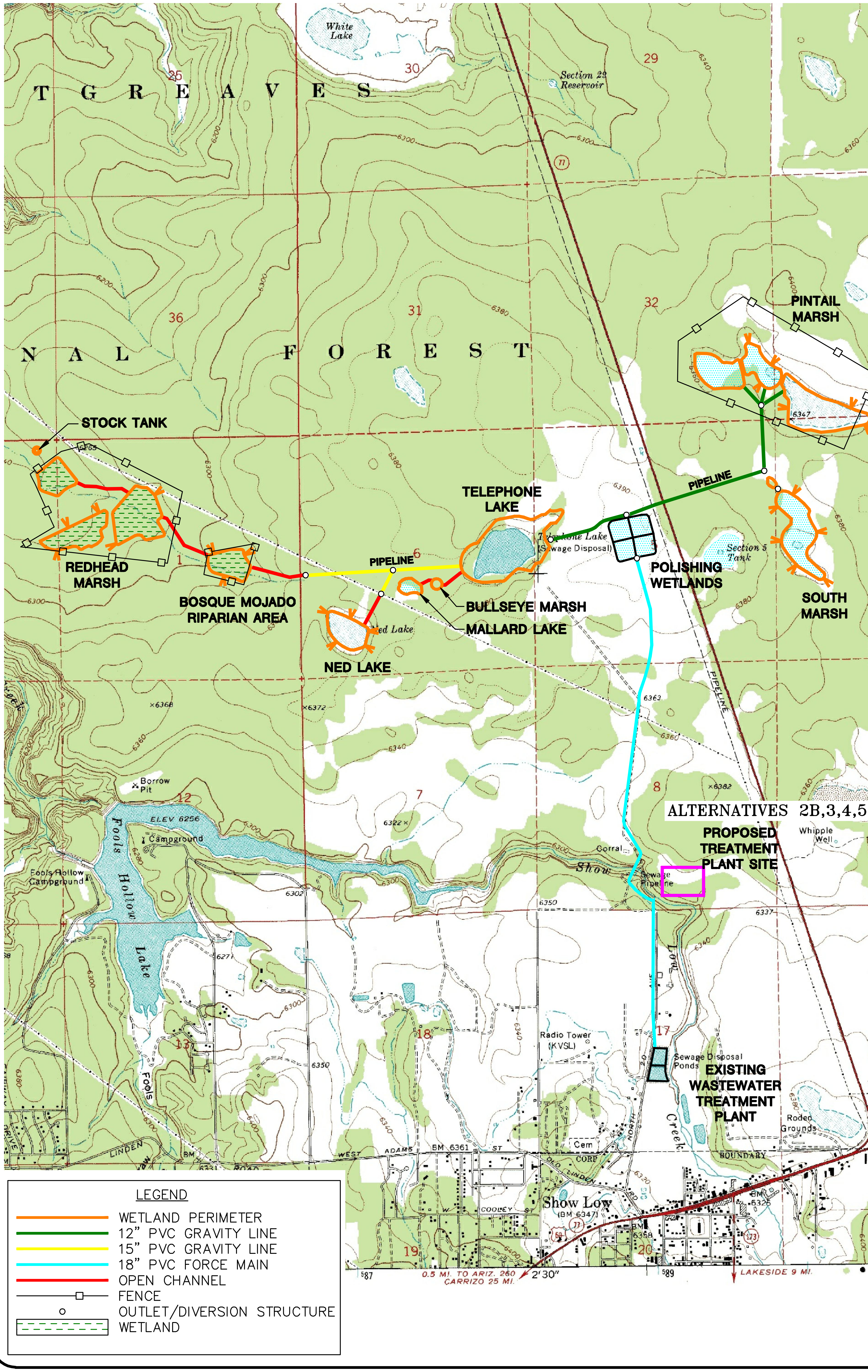


FIGURE 5-10

ALTERNATIVES 2B,3,4,5 PROPOSED WASTEWATER TREATMENT PLANT SITE

CITY OF SHOW LOW
WASTEWATER MASTER PLAN

WOOD/PATEL
MISSION: CLIENT SERVICE
PHOENIX • MESA • TUCSON

2051 W. Northern Ave.
Phoenix, AZ 85021
(602) 335-8500

N:\2011\113701 Show Low WWMP\Dwg\Exhibits\FIGURE 5-10 ALTERNATIVE WWTP SITE.dwg

5.1 Effluent Disposal

The Master Plan addresses four (4) disposal alternatives. They are:

- Discharge to the existing wetlands,
- Stream discharge,
- Reclamation/ reuse and
- Open access irrigation.

Discharge to wetlands requires a class B+ treatment as defined in the Arizona Administrative Code R18-11-3. Stream discharge, open access irrigation and reclamation/ reuse require Class A+ treatment. Treatment requirements for these uses are identified in Table 5-1.

Table 5-1 - Treatment Requirements for Various Effluent Alternatives

	RW Class[1]	Primary	Secondary	Nutrient Removal	Chemical Coagulation	Filtration	Disinfection
		A+, B+	A+, B+	A+, B+	A+	A+	A+, B+
WETLAND DISCHARGE							
Existing wetlands	B+	⌘	⌘	⌘			⌘
STREAM DISCHARGE							
Effluent Discharge	A+	⌘	⌘	⌘	⌘	⌘	⌘
RECLAMATION/REUSE							
Recreational impoundments (Fool Hollow)	A+	⌘	⌘	⌘	⌘	⌘	⌘
IRRIGATION							
Golf course irrigation	B+	⌘	⌘				⌘
Restricted access landscape irrigation	B+	⌘	⌘				⌘
Open access landscape irrigation	A+	⌘	⌘		⌘	⌘	⌘

[1]Reclaimed water classification as defined in R18-11 Article 3. Additional requirements may apply. See Appendix B, R18-11.

5.2 Alternative 1 - Existing WWTP Analysis and Improvements

The treatment capacity of the existing lagoon system was analyzed. The objective was to identify improvements that could be made with limited capital investment to meet current discharge permit requirements. Three flow conditions were examined, using the wastewater characteristics presented earlier in this report:

- Current – 1.02 MGD (nominal)
- Phase 1 – 1.75 MGD
- Phase 2 – 2.46 MGD (current permit limit)

The analysis was based on performance data from the Show Low WWTP. Aeration improvements made over the past three years have increased treatment capacity. The analysis was based on continuing those types of improvements, including new aerators and adding lagoon covers (as discussed in the Phase 2 report).

Performance analysis and predictions were made using a complete-mix biological treatment model, "Biowin." Biowin is widely used in the industry for biological treatment system analysis and design. Results from the Biowin analyses are presented and discussed in the following sections.

5.2.1 Assessment of Lagoon Performance

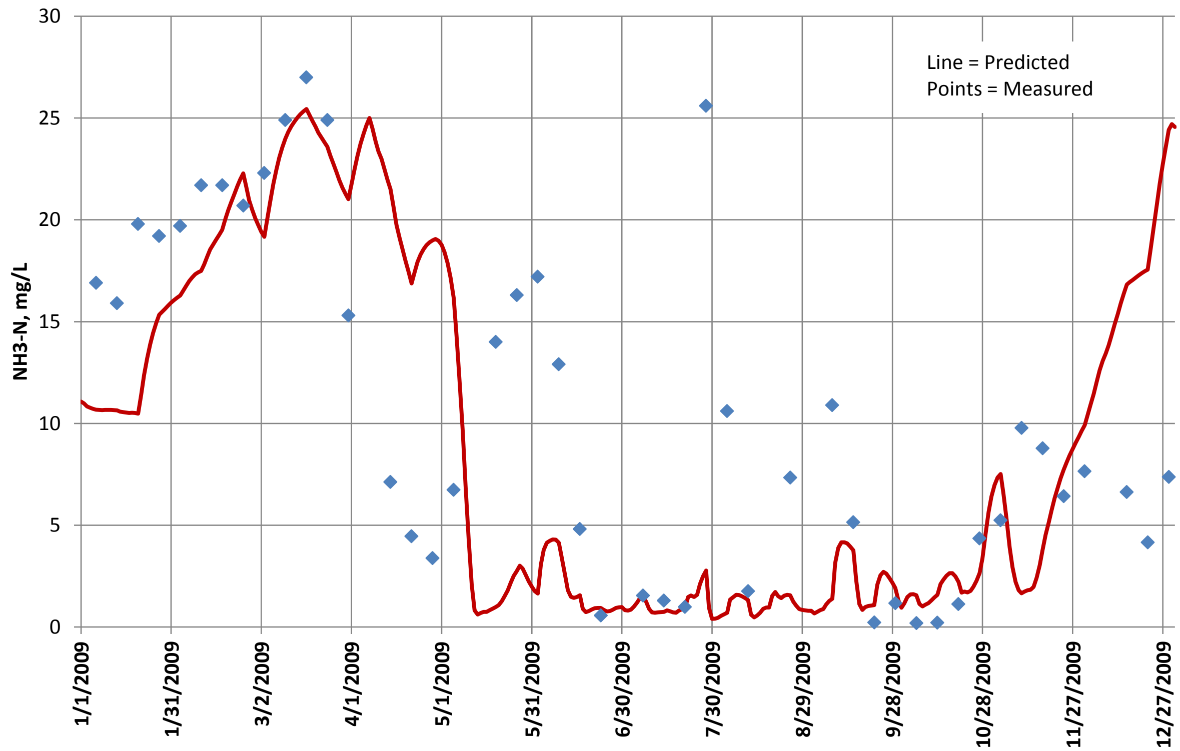
Floating aerators ("Triton" units from Aeration Industries) have been installed in Lagoon 1 over the past three years in several phases. The aeration upgrade began with installing four 20-hp aerators in March 2008. Two more were added in November 2008. Two more were added in May 2011, bringing the total to the current eight units.

Performance of the lagoons with regard to ammonia removal with improved aeration was examined using plant data and Biowin model simulations. The results for calendar year 2009 are shown in Figure 5-1. Six Triton aerators were in service throughout this period. While there is scatter in the data, the model quite clearly shows the trends in performance. The model was calibrated by adjusting one kinetic parameter, the nitrifier maximum specific growth rate. Calibration is a normal part of computer modeling. While a biological kinetic parameter was used for calibration, it is likely (although not certain) that the observed performance is due to the partial mix regime of the lagoon, rather than a true description of kinetics (although this cannot be confirmed with the available data).

Performance has been variable. This is due to several factors, the most important of which are believed to be:

- Incomplete mixing
- Inadequate oxygen transfer
- Low lagoon temperature

Figure 5-1
Predicted and Measured Lagoon 1 Ammonia Concentrations
(2009 Data)



Each of these was examined using the Biowin model and performance data collected by plant staff. In all of the following Biowin simulations, the "default" (most commonly observed in operating plants) value for the nitrifier maximum specific growth rate was used, rather than the value used to calibrate the model.

5.3 Mixing

Mixing efficiency in aerated lagoons is approximated by energy input. The distribution of that energy and how it is delivered to the water (particularly the type of aeration/mixing equipment) are key parameters in determining actual mixing efficiency. Typically, an energy input of 30 hp/MG using Triton aerators is considered necessary to get good mixing in an aerated lagoon (based on recommendations by the manufacturer, Aeration Industries International, Inc.).

In 2009 the energy intensity in Lagoon 1 was 16 hp/MG (120 hp/7.5 MG). This is well below the value typically considered to be necessary for adequate mixing. Therefore, it is likely that mixing was not complete. This is likely to account for some of the performance variability observed.

To achieve complete mixing in Lagoon 1, a minimum of twelve 20-hp Triton aerators would be required. Oxygen transfer requirements might necessitate more aerators, as discussed later.

5.4 Oxygen Transfer

The nominal oxygen transfer requirement for BOD_5 removal is typically about 1.0 lb O2/lb BOD. Similarly, the nominal oxygen transfer requirement for NH_3-N oxidation is about 4.6 lb O2/lb NH_3-N. Using these values, the following approximate values for "field" oxygen transfer (oxygen that must be delivered to the wastewater) were calculated:

- Current (1.02 MGD) FOTR = 159 lb O_2/hr[1]
- Phase 1 (1.75 MGD) FOTR = 272 lb O_2/hr
- Phase 2 (2.46 MGD) FOTR = 383 lb O_2/hr

Based on measurements made by operating staff, the clean water, or "standard," oxygen transfer rate (SOTR) of the Triton aerators was estimated to be 2.7 lb O2/hp-hour. This value was used for analyzing the capacity of the existing wastewater lagoon and future WWTP needs.

[1] [(275 mg/L BOD) x (1.0 mg O_2/mg BOD) + (50 mg/L TKN x 0.75 mg NH_3-N/mg TKN) x 4.6 mg O_2/mg NH_3-N)] x (1.02 MGD) x 8.34 / (24 hr/day) = 151 lb O_2/hr

The ratio of field oxygen transfer rate (FOTR) to SOTR depends on a number of factors and varies with time. It is approximately 0.4 for the conditions at Show Low. Using this value the SOTR requirements are approximately:

- Current (1.02 MGD) SOTR = 397 lb O2/hr
- Phase 1 (1.75 MGD) SOTR = 680 lb O2/hr
- Phase 2 (2.46 MGD) SOTR = 956 lb O2/hr

Using these values and the estimated SOTR of the Triton aerators, the following results were obtained:

- Current Requirement = Eight each 20-hp Triton aerators (all in Lagoon 1; a minimum of twelve aerators will be necessary to meet mixing requirements)
- Phase 1 Requirement = Thirteen each 20-hp Triton aerators (all in Lagoon 1)
- Phase 2 Requirement = Twenty-five each 20-hp Triton aerators (thirteen in Lagoon 1; five in Lagoon 2 for oxygen transfer but twelve to meet mixing requirements)

Note that only Lagoon 1 would require aeration for treatment in Phase 1. To operate the plant under Phase 2 flow conditions, both lagoons would require aeration. In both cases a portion of Lagoon 2 would be used as a settling zone. This zone would require periodic dredging for solids removal.

5.5 Lagoon Temperature

Biowin simulations were run using the numbers of aerators required as estimated above. With aerators in Lagoon 1 only, that is the only lagoon in which significant treatment will occur. Lagoon 2 would serve as a settling tank. The modeling results for ammonia concentrations in Lagoon 1 are shown in Figures 5-2 through 5-4.

Figure 5-2
Predicted Lagoon 1 Ammonia Concentrations
(1.02 mgd, 12 ea 20-hp Triton Aerators in Lagoon 1, Not Covered)

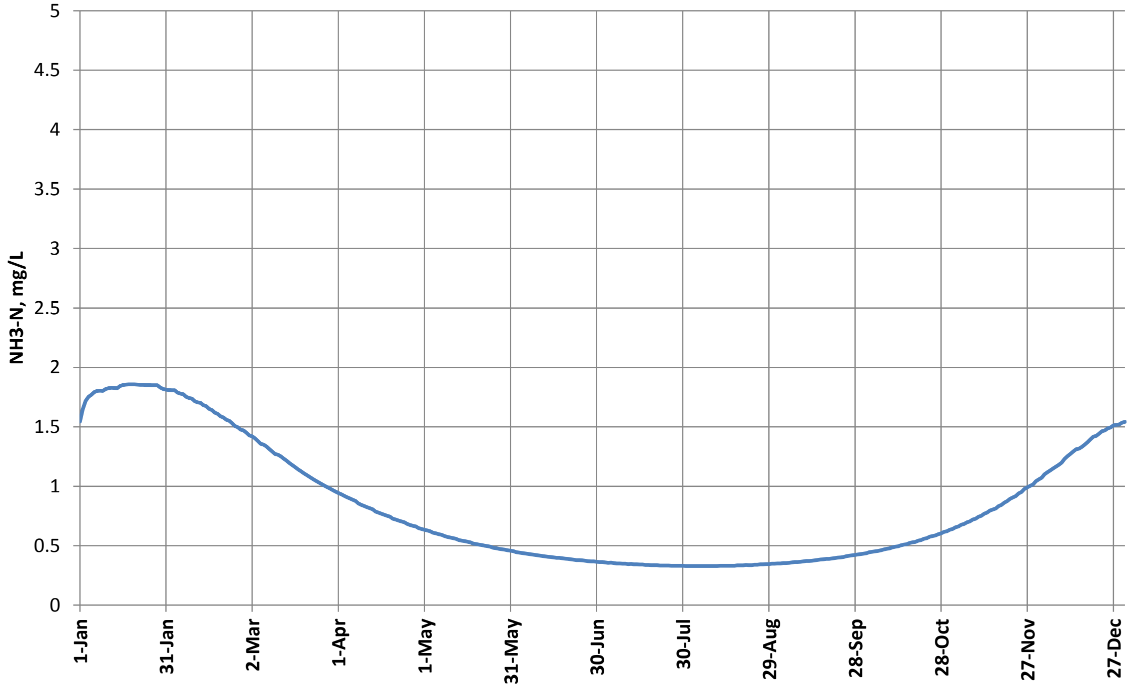
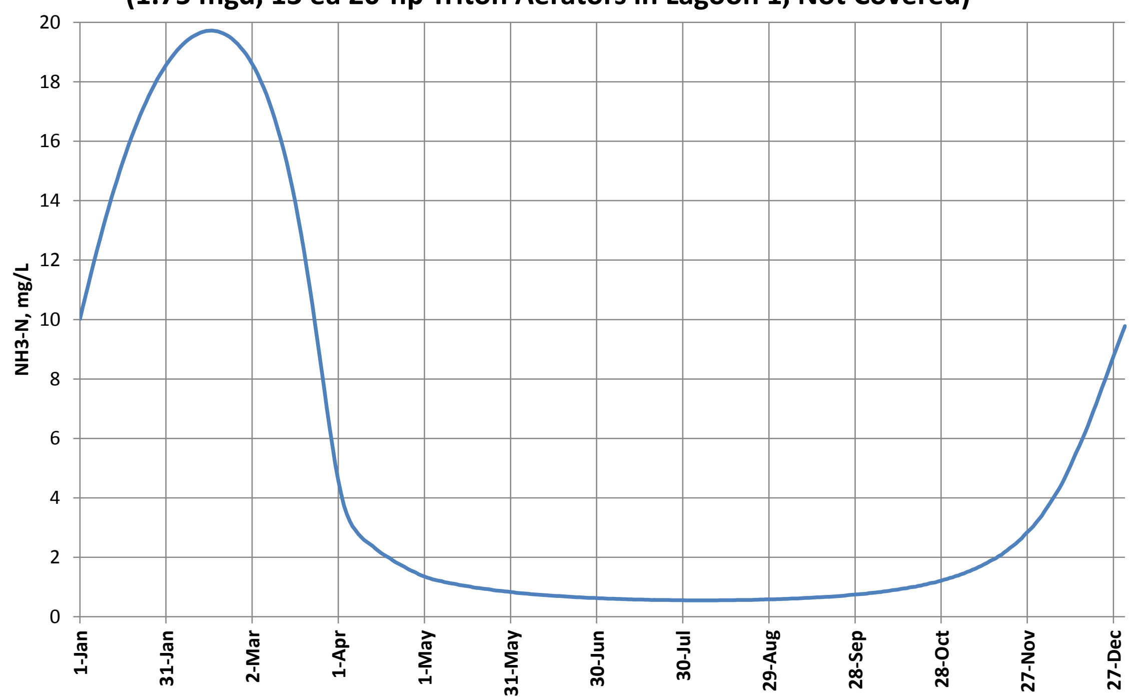


Figure 5-3
Predicted Lagoon 1 Ammonia Concentrations
(1.75 mgd, 13 ea 20-hp Triton Aerators in Lagoon 1, Not Covered)



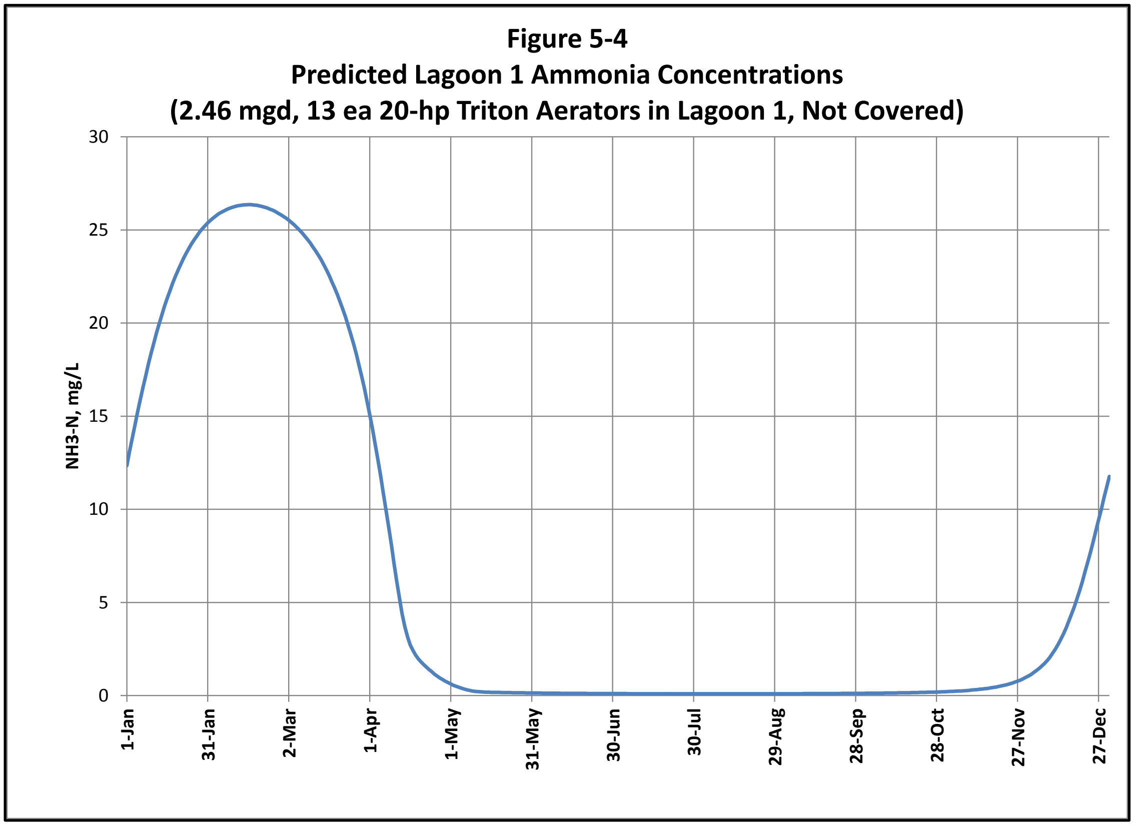
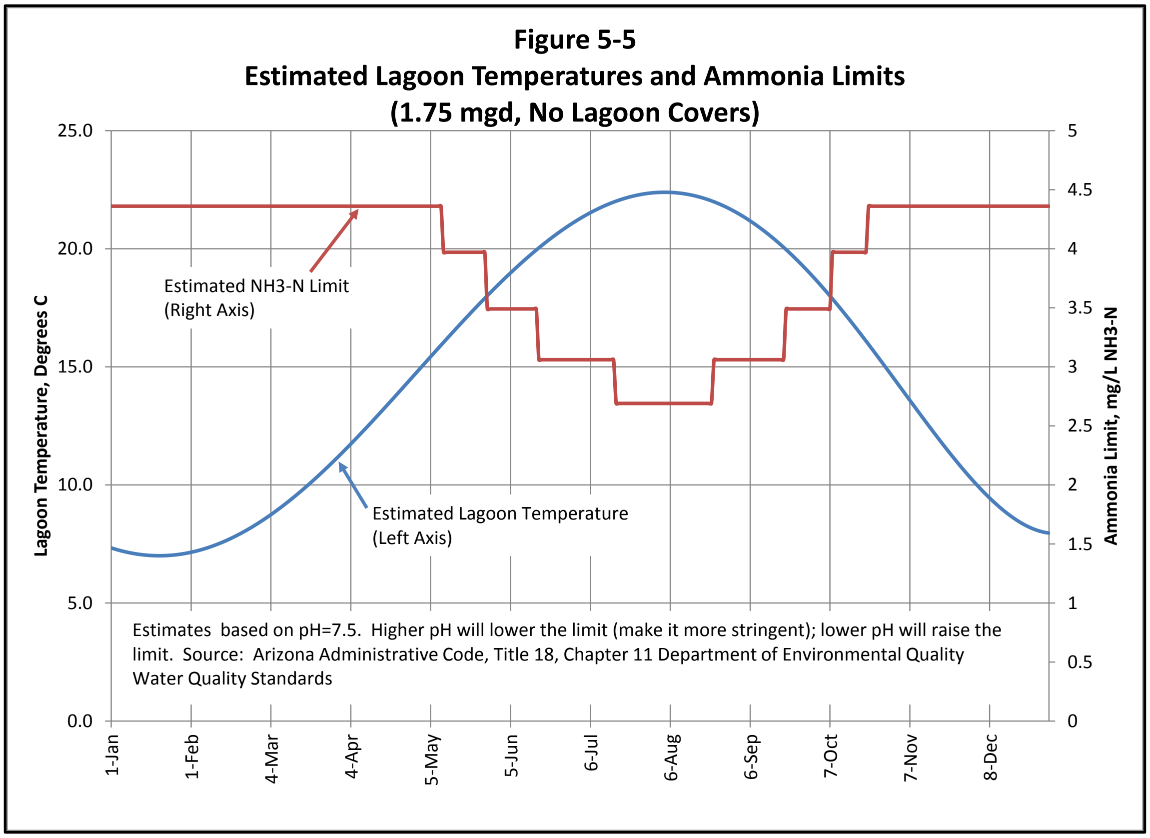


Figure 5-5 shows the estimated lagoon temperatures and corresponding NH_3-N limits, based on an effluent pH of 7.5. The limits become more stringent as pH increases. Assuming Lagoon 1 is well-mixed and does not develop algae, this pH value is considered conservatively high. Without good mixing, algae will develop and will cause the pH to rise and fall diurnally. Similarly, the limits become more stringent as temperature increases so the estimated lagoon temperature was used. Further cooling in the winter is likely and final discharge temperatures will be lower.

Comparing Figure 5-5 with the modeling results shows that the NH_3-N limits will not be met in either Phase 1 or Phase 2 with aerator improvements alone. Low temperatures during the winter inhibit nitrification.

Performance predictions were made assuming lagoon covers were installed to reduce heat loss (only Lagoon 1 covered for Phase 1, both lagoons covered for Phase 2). The results of these modeling runs are shown in Figures 5-6 through 5-8. These modeling runs assume a minimum “effective R” value for the covers of approximately 4. The actual cover would need an R value of 5 or greater to compensate for heat losses through the openings required for the aerators.

These modeling results indicate that covering Lagoon 1 for heat retention, together with aeration improvements, should allow the NH_3-N limits to be met year-round at Phase 1 flows. These improvements will not be sufficient to meet the limits at Phase 2 flows.



An option for further improving lagoon performance would be to recycle flow from Lagoon 2 to Lagoon 1. This would have four potential benefits:

- Nitrifier "seeding"
- Denitrification for nitrate control
- Denitrification for total oxygen demand reduction
- Denitrification for alkalinity control

First, it would return a population of nitrifiers to the front of the plant. Nitrification would then be distributed through the plant. This would increase the total nitrification potential for the plant. Otherwise, at Phase 2 flows nitrification would occur only in Lagoon 2.

Second, it would return a flow of nitrified water to the head of the plant where it could be "denitrified." Show Low has nitrate limits for monitoring wells in the vicinity of the polishing ponds. As flows increase the ability of the polishing ponds to control nitrate will diminish. Denitrification at the lagoons would lessen the impact of nitrogen on the polishing ponds.

Third, returning nitrified water to the head of the plant for denitrification would reduce the net oxygen requirement to be supplied by aeration. The denitrification process removes BOD_5 which does not require oxidation with dissolved oxygen. This reduces both the amount of aeration equipment required (capital cost) and the power requirement for aeration (operating cost).

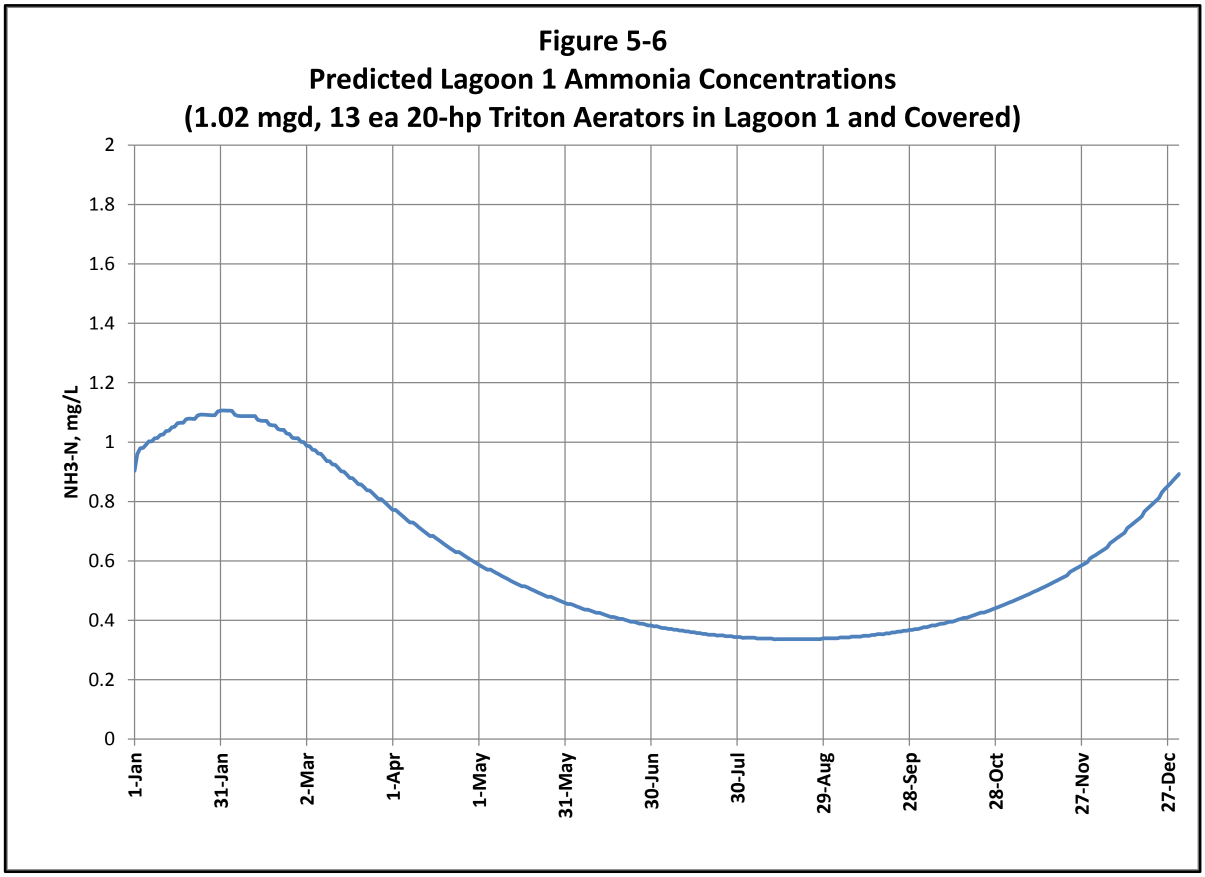


Figure 5-6
Predicted Lagoon 1 Ammonia Concentrations
(1.02 mgd, 13 ea 20-hp Triton Aerators in Lagoon 1 and Covered)
NH3-N, mg/L
2
1.8
1.6
1.4
1.2
1
0.8
0.6
0.4
0.2
0
1-Jan
31-Jan
2-Mar
1-Apr
1-May
31-May
30-Jun
30-Jul
29-Aug
28-Sep
28-Oct
27-Nov
27-Dec

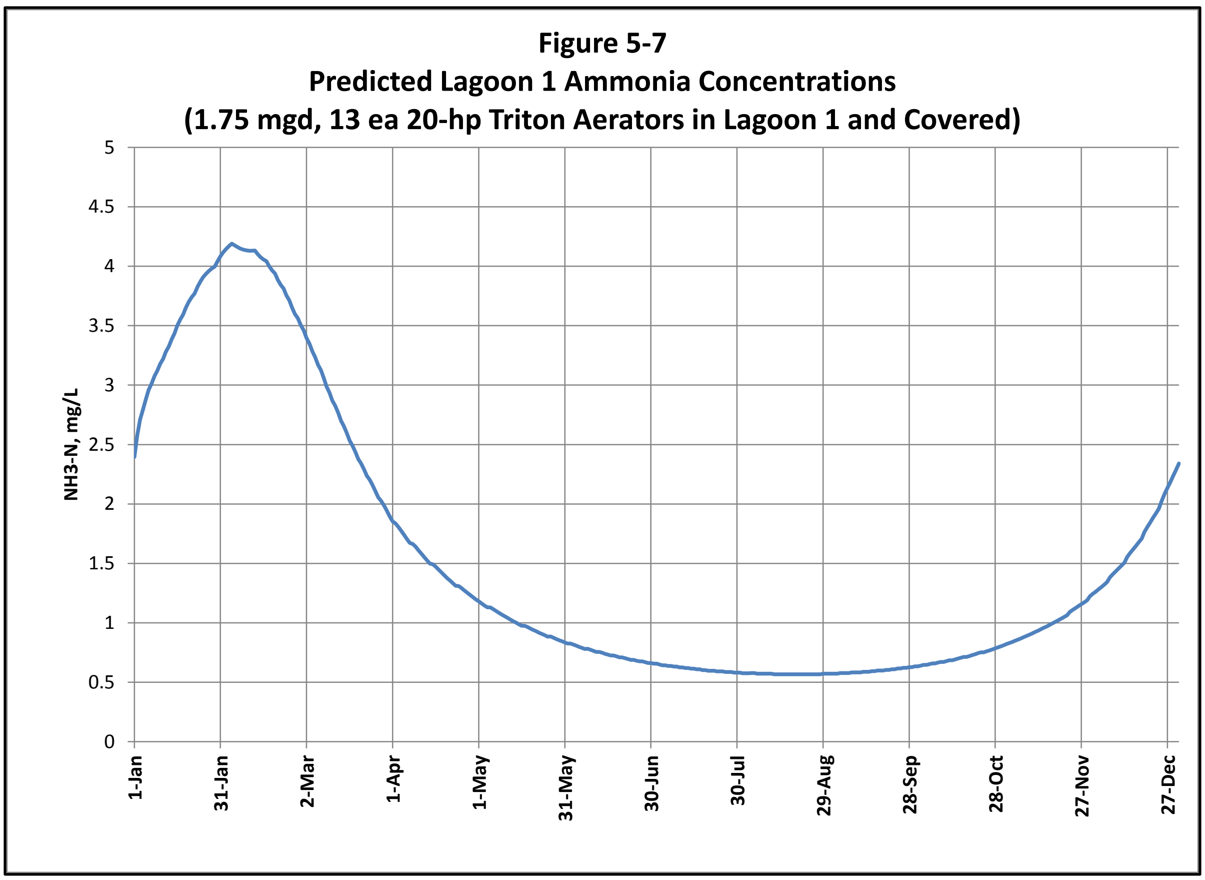
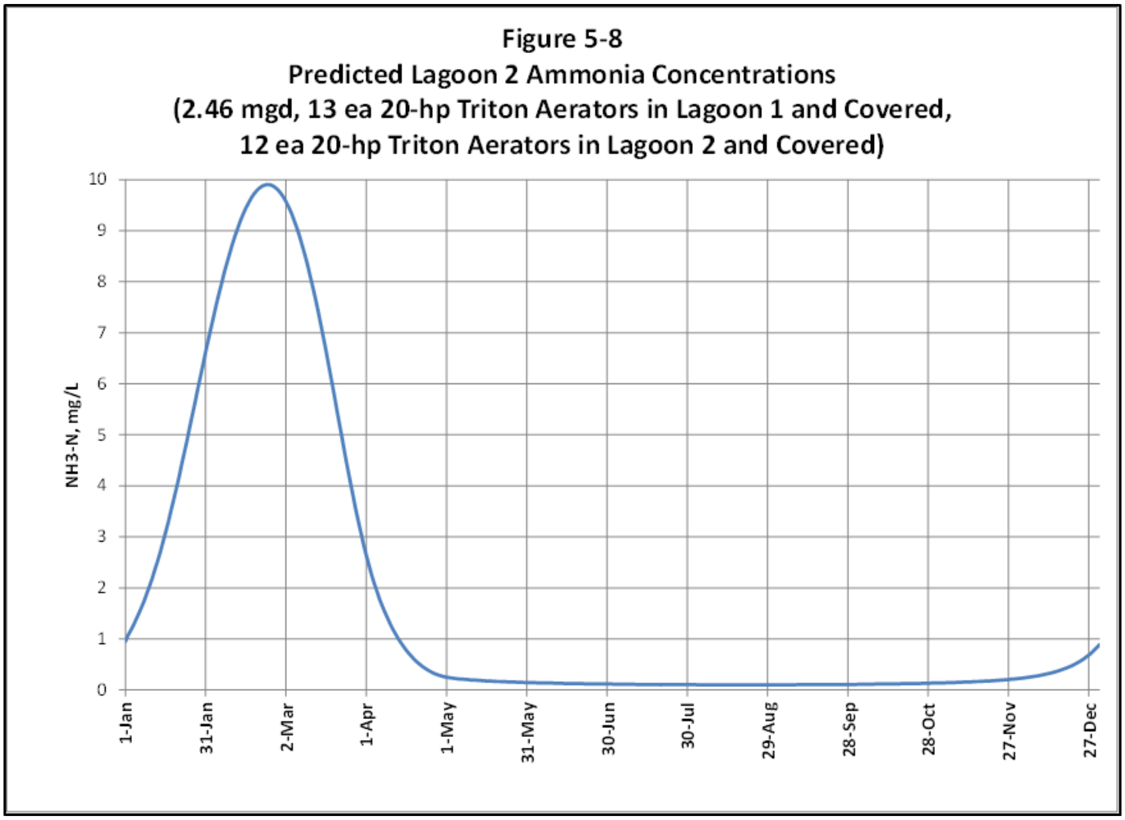


Figure 5-7
Predicted Lagoon 1 Ammonia Concentrations
(1.75 mgd, 13 ea 20-hp Triton Aerators in Lagoon 1 and Covered)
NH3-N, mg/L
5
4.5
4
3.5
3
2.5
2
1.5
1
0.5
0
1-Jan
31-Jan
2-Mar
1-Apr
1-May
31-May
30-Jun
30-Jul
29-Aug
28-Sep
28-Oct
27-Nov
27-Dec



Finally, denitrification recovers alkalinity. The nitrification process consumes alkalinity. If the alkalinity drops too low the nitrification process is inhibited.

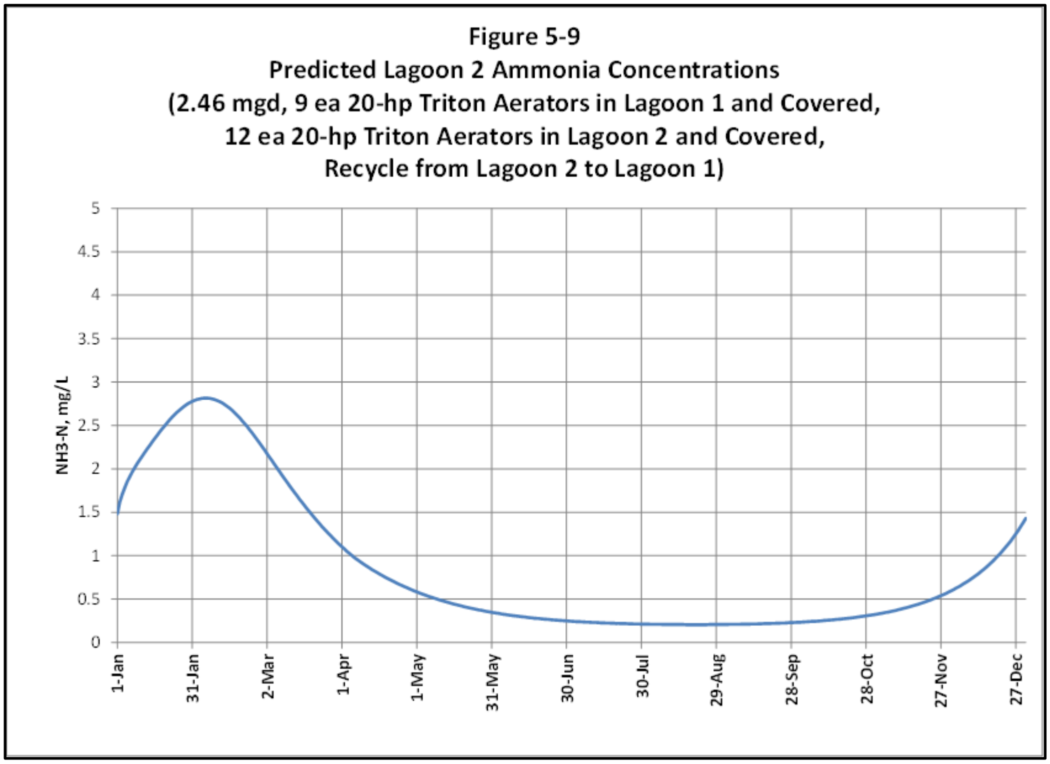
Figure 5-9 shows the modeling predictions for NH_3-N in Lagoon 2. Again, these predictions are based on:

- Covering both lagoons for heat retention
- Installing a total of twenty-one aerators in the two lagoons (nine in Lagoon 1 and twelve in Lagoon 2)
- Recycling flow from Lagoon 2 to Lagoon 1 (the modeling predictions are based on recycling 2.46 MGD)

As indicated by the results in Figure 5-9, making the improvements described has the potential to allow the existing lagoon system to meet the nitrogen limits in the current discharge permit for the plant. Importantly, these improvements could be phased. This would allow the predictions presented here to be verified with operating data.

5.6 Recommended Improvement Program

Based on the WWTP analysis presented in the previous sections, the following phased approach for WWTP improvements is recommended under Alternative 1:



Phase 1:

1) Install an additional five each 20-hp Triton aerators in Lagoon 1 (bringing the total to thirteen in that lagoon)
2) Arrange those aerators to optimize mixing and oxygen transfer
3) Install a floating cover over the entire lagoon surface to retain heat
4) Baffle Lagoon 2 to create a settling zone of approximately one million gallons for solids removal (not discussed in this report but a critical improvement requirement)
5) Upgrade the headworks with a new 6-mm screen (not discussed in this report but a critical improvement requirement to protect the aerators)

Phase 2:

1) Relocate four 20-hp Triton aerators from Lagoon 1 to Lagoon 2 (reducing the number in Lagoon 1 to nine)
2) Install an additional eight 20-hp Triton aerators in Lagoon 2 (bringing the total to twelve in that lagoon)
3) Install a baffle in Lagoon 1 to create a 2.5 million gallon volume at the inlet to the lagoon, to be operated as an anoxic zone
4) Install three each 10-hp Triton mixers in the anoxic zone (for mixing only)
5) Arrange all aerators to optimize mixing and oxygen transfer
6) Install a floating cover over the entire surface of Lagoon 2 to retain heat
7) Install a recycle pump station to return flow from the mixed section of Lagoon 2 to the influent to Lagoon 1

Phase 1 improvements should provide treatment capacity to at least 1.75 MGD. While operating with these improvements, assessments should be made of all the parameters identified earlier as affecting performance. There are no improvements required for the polishing wetlands in phases 1 or 2.

5.7 Preliminary Estimate of Probable Construction Cost – Alternative 1

Table 5-2 summarizes the Preliminary Estimate of Probable Construction Cost for constructing the improvements described above for Phase 1. Table 5-3 summarizes the Preliminary Estimate of Probable Construction Cost for constructing the additional improvements described above for Phase 2.

The preliminary estimate of probable construction and engineering cost for Phase 1 is $1.8 million. The preliminary estimate of probable construction and engineering cost for Phase 2 is $2 million. The higher estimated cost for Phase 2 is due to the requirement to use both lagoons. Some required improvements for Phase 1 are already in place, reducing the additional cost for that phase.

Both of these costs are less than the cost of constructing a new WWTP. However, a point will be reached at which improvements of the existing WWTP will not meet the treatment requirements. This may be before Phase 2 flows are reached, although this analysis indicates that the plant will be capable of treating those flows.

Table 5-2 – Alternative 1: Preliminary Estimate of Probable Construction Cost Phase 1 [1]

Flow to 1.75 MGD				
Description	**Number**	**Description**	**Unit Cost**	**Total Cost**
Mechanical Bar Screen	1	6 mm (Phase 2 Capacity)	$150,000	$150,000
R-5 Cover	1	115,000 SF	$4.50/SF	$518,000
Aerators	5	20-hp Triton	$30,000	$150,000
Baffle	1	300 LF	$30/LF	$9,000
Electrical Upgrades	1	Allowance	$100,000	$100,000
Equipment Installation	1	Allowance	50%	$465,000
			Lagoon Improvements Costs	$1,392,000
			15% Engineering/Admin	$209,000
			25% Contingency	$348,000
			TOTAL	**$1,949,000**

[1]All costs 2011 dollars.

Table 5-3 – Alternative 1: Preliminary Estimate of Probable Construction Cost Phase 2[1]

Flow to 2.46 MGD				
Description	**Number**	**Description**	**Unit Cost**	**Total Cost**
Baffle	1	300 LF	$30/LF	$9,000
Mixers	3	10-hp Triton	$20,000	$60,000
R-5 Cover	1	115,000 SF	$4.50/SF	$518,000
Aerators	8	20-hp Triton	$30,000	$240,000
Recycle Pump Station	1	1,700 gpm	$75,000	$75,000
Recycle Pipeline	1	700 LF	$30/LF	$21,000
Electrical Upgrades	1	Allowance	$100,000	$100,000
Equipment Installation	1	Allowance	50%	$511,000
			Lagoon Improvements Costs	$1,534,000
			15% Engineering/Admin	$230,000
			25% Contingency	$383,500
			TOTAL	**$2,147,000**

[1]Assumes all Phase 1 Upgrades in place. All costs 2011 dollars.

5.8 Alternatives 2 through 5 - Preliminary Screening of Alternatives

The treatment processes must comply with the Arizona Department of Environmental Quality (ADEQ) Best Available Demonstrated Control Technology (BADCT) requirements. Treatment alternatives also focused on pursuing the following objectives:

- Alternatives that emphasize discharge to wetlands (B+ effluent).
- Ability to perform open irrigation (A+ effluent).
- Ability for stream discharge (A+ effluent).
- Alternatives that maximize the use of residual solids.
- Alternatives that minimize chemical consumption.
- Alternatives that progress from logical short-term solutions to logical long-term solutions without expensive mid-course corrections.
- Alternatives that are cost effectively modified to meet new standards.

The following alternatives were eliminated from further consideration because they do not support the objectives:

- Fixed growth wastewater treatment processes (trickling filter, biological tower and rotating biological contractor) because they do not comply with Best Available Demonstrated Control Technology (BADCT).
- Suspended growth processes that utilize methanol feed to partially remove nitrogen because of high chemical costs.
- Sole use of chlorination disinfection for class A+ effluent because it is difficult to comply with BADCT.
- Anaerobic digestion because of high capital cost and operational complexity.
- Chemical stabilization of sludge because of high capital and chemical costs.
- Sludge drying beds because of the large land requirements, odor generating potential and limitations during winter months.
- Separate selenium removal because it is anticipated removal will occur within the treatment processes evaluated.

5.9 Alternatives 2 through 5 - Treatment Process Alternatives Evaluated

The following treatment processes were evaluated:

- Suspended growth secondary treatment (activated sludge).
- Biological removal of total nitrogen concentration to less than 10 mg/1 as N.
- Combination of biological and chemical phosphorus removal for an effluent of < 0.1 mg/L.
- Filtration followed by ultraviolet light irradiation as the disinfection method for A+ effluent.

The following four modifications of the activated sludge process were selected for this Master Plan:

1. Biolac ® extended aeration ,
2. extended aeration activated sludge (EAAS),
3. sequencing batch reactors (SBR) and
4. membrane bioreactors (MBR).

The Biolac system is unique from the other three alternatives in that it would be constructed on the existing site, taking advantage of the configuration of the existing lagoons.

All alternatives would have similar preliminary treatment (flow measurement, solids screening, and grit removal), similar disinfection and similar sludge processing facilities. Class A+ effluent is not proposed for the Biolac, however it can be expanded to include it at a later date. The EAAS and SBR will have similar tertiary treatment (filtration) for class A+ effluent. The MBR will not require separate filtration as the membranes provide the filtration.

5.9.1 Biolac® Extended Aeration

The key parameters which distinguish the Biolac extended aeration from other activated sludge processes are a hydraulic retention time of over 36 hours and a solids retention time (SRT) of over 30 days. Aeration is provided by moving chain surface floating header pipes connected to submerged fine bubble diffusers.

The moving aeration chain aeration devices provide both the aeration and mixing energy to keep the mixed liquor in suspension. The aerators may be turned on, turned off or modulated to adjust the dissolved oxygen within the basin in order to achieve biological nutrient removal.

Following is a summary of the advantages and disadvantages of the Biolac process.

Advantages:

- It is a stable process that is able to adapt to variations in wastewater flow and waste strength.
- Biolac plants may be operated with minimal operator attention. It may be operated to achieve biological nutrient removal.
- It can be constructed in the existing lagoon basins.
- Total effluent nitrogen concentration of less than 10 mg/1can be achieved in a single aeration basin.

Disadvantages

- A separate anaerobic stage will be required if the system is to be expanded for biological phosphorus removal.
- Biolac plants have a large footprint, which is costly in terms of space utilization.

5.9.2 Extended Aeration Activated Sludge (EAAS)

The key parameters which distinguish extended aeration from other activated sludge processes are hydraulic retention time of approximately 24 hours and long solids retention time (SRT) in the aeration basin. Aeration can be provided by either surface mounted aerators or submerged diffusers. Surface aerators may be mounted horizontally across the basin, vertically mounted or floating. Submerged diffusers may be either fixed or suspended.

Aeration devices provide both the aeration and mixing energy to keep the mixed liquor in suspension. The aerators may be turned on, turned off or modulated to adjust the dissolved oxygen within the basin in order to achieve biological nutrient removal. Separate mixing devices would be required for the biological nutrient removal processes.

Several processes may be classified as extended aeration activated sludge (EAAS). These include oxidation ditches, and Aeromod's Sequox processes, among others. This study does not provide separate evaluations for each extended aeration process modification. Rather, a generic extended aeration activated sludge plant is presented in this evaluation.

Following is a summary of the advantages and disadvantages of the EAAS process.

Advantages

- EAAS is a stable process that is able to adapt to variations in wastewater flow and waste strength.
- EAAS plants may be operated with minimal operator attention. EAAS may be operated to achieve biological nutrient removal.
- Numerous manufacturers produce equipment for EAAS plants.

Disadvantages

- A separate anoxic stage will be required prior to the aeration stage to achieve total nitrogen effluent concentration of 10 mg/1.
- A separate anaerobic stage will be required prior to aeration for biological phosphorus removal.
- EAAS plants have a relatively large footprint, which is costly in terms of construction cost and space utilization.

5.9.3 Sequencing Batch Reactor (SBR)

An SBR system is an activated sludge process that treats wastewater in measured batches rather than as a continuous flow. Several reactors are provided to operate in parallel to treat the continuous flow of wastewater received at the WRF.

Typically, each reactor operates in a sequence of six phases with a hydraulic retention time of 4 to 6 hours for each sequence. The first phase is the "fill" phase in which a tank receives wastewater until it is nearly full. The next phase is the "mixed fill" phase in which mixing occurs for a period of time during the fill phase.

The third phase is the "aeration" phase in which oxygen is added and the mixed liquor is kept in suspension by the air or mechanical mixing. The fourth phase is the "settling" phase in which the mixing is stopped and the tank functions as a final clarifier. During this phase the sludge settles to the bottom of the tank while the clarified effluent rises to the upper portion of the tank.

The "discharge" phase is fifth, in which the effluent is decanted from the top while the sludge remains in the basin. The last phase is the "idle" phase in which the tank waits for the next "fill" phase. During the "idle" phase, a portion of the sludge remaining in the tank is wasted. A key element of the SBR process is that the tank is never completely emptied, but rather a portion of the settled sludge is left in the tank as a seed for the next phase.

Advantages

- The SBR process is stable and can adapt to variations in wastewater flow and waste strength.
- The SBR process is easily expanded in phases.
- The SBR process can be controlled to provide partial nitrogen removal.
- The process can be controlled to achieve biological phosphorus removal.
- Final clarifiers are not needed. This reduces capital and operation and maintenance (O&M) costs.

- Return activated sludge and recycle pump stations are not needed. This reduces capital and O&M costs.
- Land area for SBR may be less than for EAAS.
- Several manufacturers market SBR equipment.

Disadvantages

- An equalization basin is required after SBRs to provide a constant flow rate to filters and disinfection facilities.
- Control of Nocardia bacteria is difficult, so foam often accumulates on the surface of the basin. This foam may reach the effluent.

5.9.4 Membrane Bioreactor (MBR)

The MBR is a suspended growth-activated sludge system that utilizes micro porous membranes immersed within the aeration basin for solid/liquid separation. The mixed liquor suspended solids concentration within the aeration basin is greater than other activated sludge processes; however the volume of the basins is less than other activated sludge processes, thus allowing for a physically smaller facility. The aeration basin is usually sized to provide a hydraulic retention time of approximately 4 to 6 hours. Aeration is provided by fine bubble diffusers and blowers. The diffusers provide air for scouring, mixing and cellular activity.

The MBR process can be designed to meet project-specific nutrient removal requirements. Anoxic zones before or after the aerobic treatment may be used for nitrogen removal. Anaerobic zones may be used for biological phosphorus removal.

Membranes, rather than clarifiers, provide the water/solids separation. Membranes with numerous pore spacing are manufactured for differing objectives. The typical pore opening size for wastewater treatment membranes is 0.1 to 0.5 millimeters (mm).

Two main types of membranes are utilized in wastewater treatment. They are flat sheet and hollow pore. The decision of which technology to employ is made during the preliminary design of the wastewater treatment plant.

The primary operational challenge with membranes is fouling. Membrane maintenance requires the elimination of fouling through use of such techniques as acid wash, flexing and backwashing. These operations require their own equipment and operational processes.

Advantages

- High quality effluent is achievable through tailor made biological processes.
- It is a stable process that is able to adapt to variations in wastewater flow and waste strength.
- It is readily adaptable to biological nutrient removal
- Computerized process control is easily achieved.

Disadvantages

- Capital and operational costs typically exceed the cost of conventional processes.
- Process requires high quality preliminary treatment.

5.10 Alternatives 2 through 5 - Treatment Process Components

Below is a description of the components within the Biolac, Extended Aeration Activated Sludge (EAAS), Sequencing Batch Reactor (SBR) and Membrane bioreactor (MBR) processes which are evaluated for this study. Figures 5-11, 5-12, 5-13 and 5-14 present diagrams for these processes.

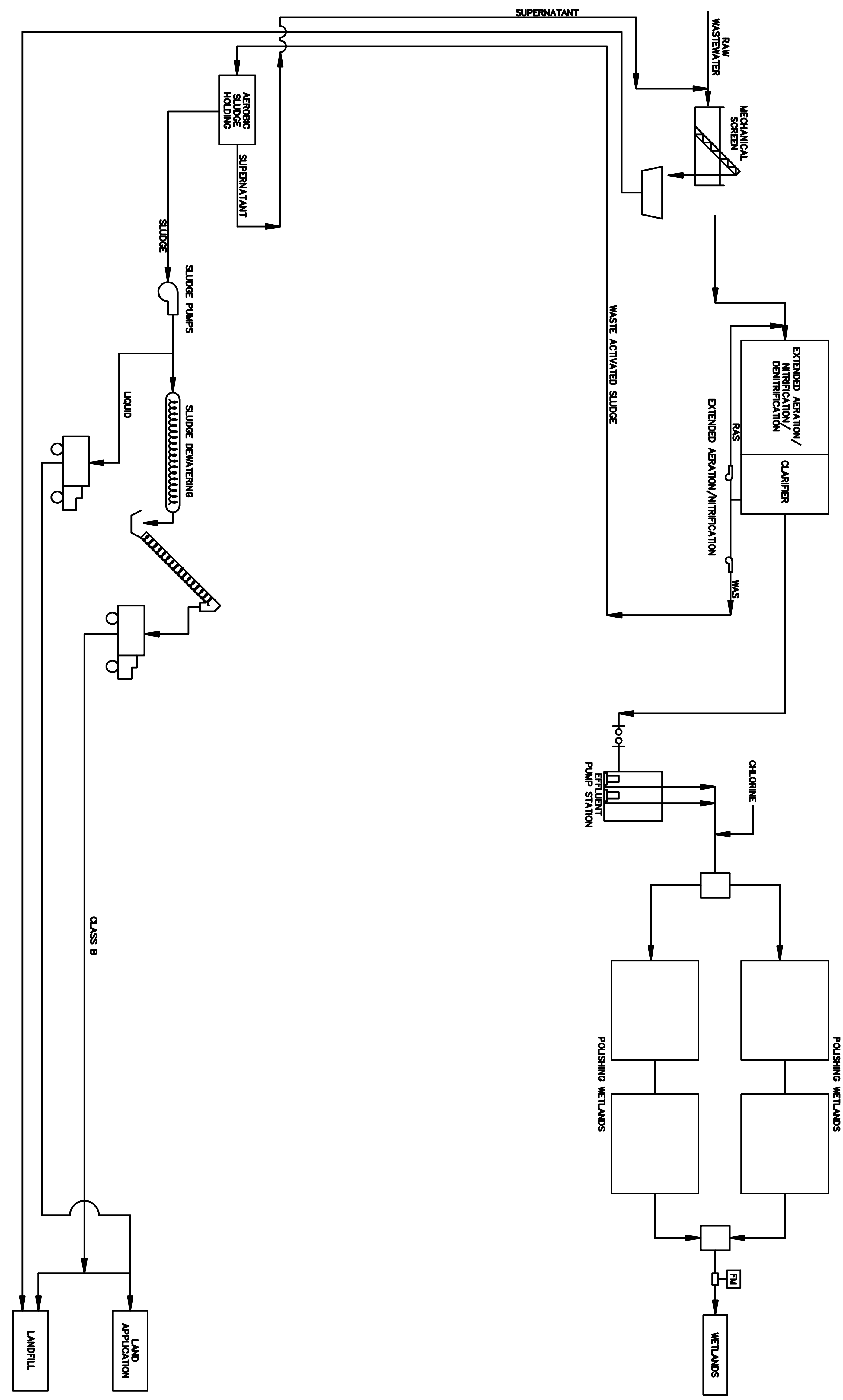


FIGURE 5-11	BIOLAC PROCESS FLOW DIAGRAM	CITY OF SHOW LOW WASTEWATER MASTER PLAN

WOOD/PATEL
MISSION: CLIENT SERVICE
PHOENIX • MESA • TUCSON
2051 W. Northern Ave.
Phoenix, AZ 85021
(602) 335-8500

N:\2011\113701 Show Low WWMP\Dwg\Exhibits\ FIGURE 5-12 EAAS FLOW DIAGRAM.dwg

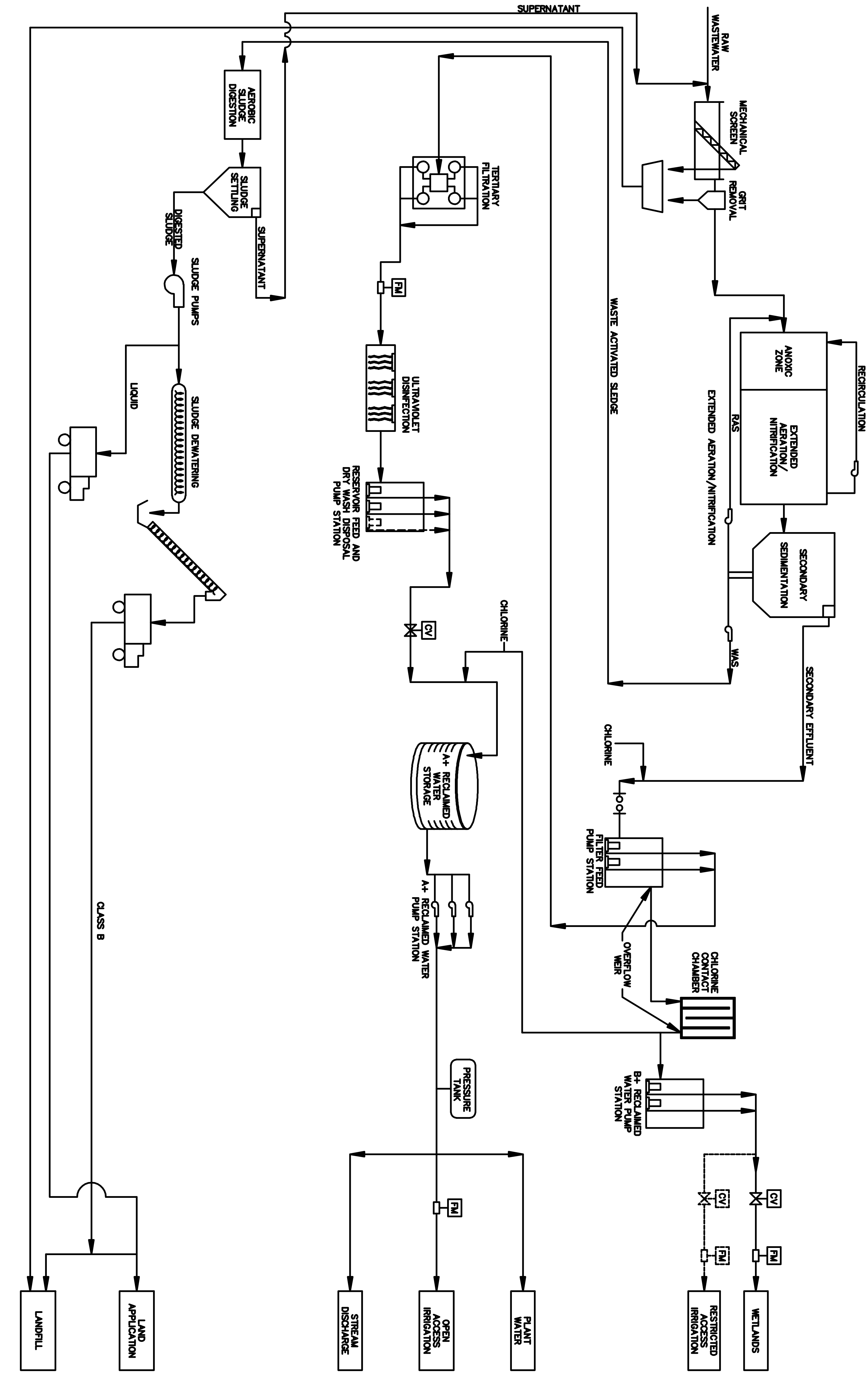


FIGURE 5-12

EXTENDED AERATION ACTIVATED SLUDGE (EAAS)
PROCESS FLOW DIAGRAM

CITY OF SHOW LOW
WASTEWATER MASTER PLAN

WOOD/PATEL
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Phoenix, AZ 85021
(602) 335-8500

N:\2011\113701 Show Low WWMP\Dwg\Exhibits\ FIGURE 5-13 SEQUENCING BATCH REACTOR.dwg

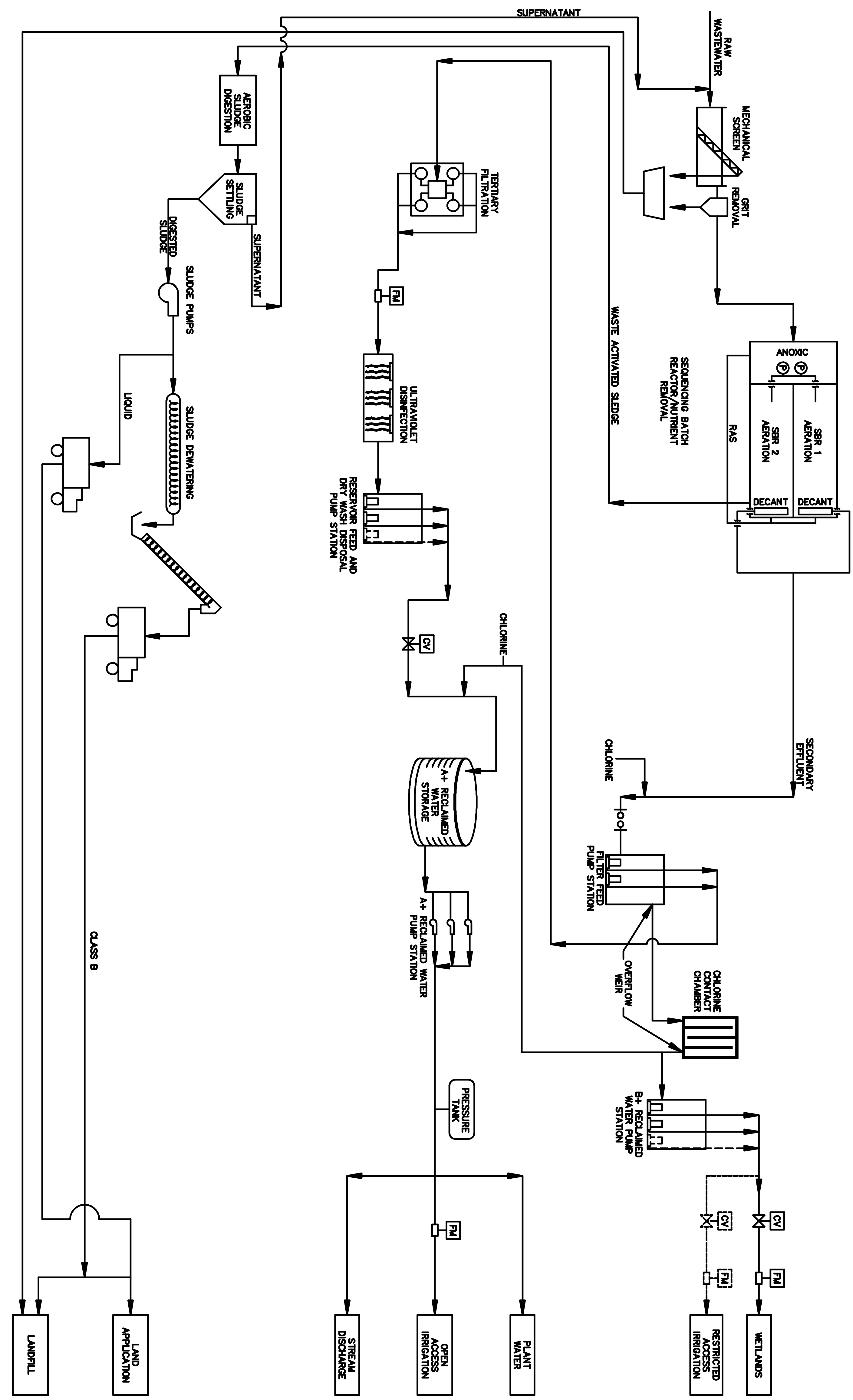

FIGURE 5-13

SEQUENCING BATCH REACTOR (SB2)

CITY OF SHOW LOW
WASTEWATER MASTER PLAN

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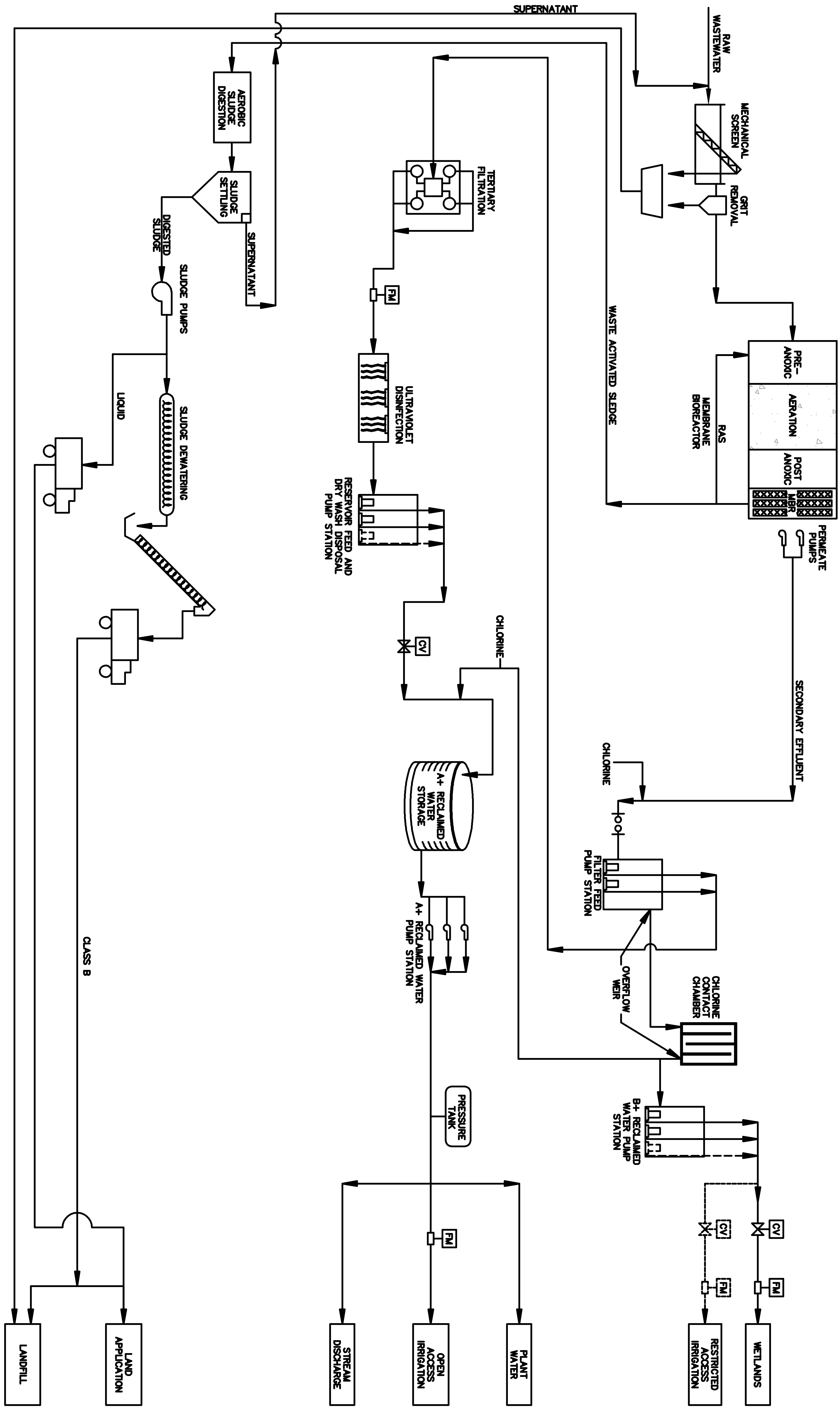

FIGURE 5–14

MEMBRANE BIOREACTOR (MBR)

CITY OF SHOW LOW
WASTEWATER MASTER PLAN

WOOD/PATEL
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Phoenix, AZ 85021
(602) 335-8500

5.10.1 Biolac

5.10.1.1 Headworks

Raw wastewater would enter the existing facility headworks. The existing coarse mechanical bar screen will be replaced by a fine screen with solids dewatering/compaction capability. Preliminary screening would remove all solids greater than 6 mm. A septage receiving station will be added for receiving septage from septage haulers.

5.10.1.2 Biological Treatment (BOD_5 and Nitrogen)

From the headworks, the wastewater would flow to the Biolac basin where it would be treated for BOD_5 and nitrogen removal. Nitrification and denitrification would biologically reduce the total nitrogen to less than 10 mg/L. Oxygen would be provided by fine bubble diffusers. Table 5-4 presents preliminary design criteria for the Biolac treatment unit.

Table 5-4. Biolac Design Criteria

Number of Treatment Trains	1
Basin Side water Depth, feet	10
Mixed Liquor Suspended Solids, mg/L	3,000
Solids Retention Time, days	30+
Oxygen Requirement for BOD_5 Removal, lb O_2/lb_R	1.5
Oxygen Requirement for TKN Oxidized, lb O_2/lb_R	4.6
Alpha/Beta	.7/.95
Average Day, Peak Hour AOR/SOR, lb/hr	400/600

5.10.1.3 Phosphorus removal

Phosphorus removal is not required under the current and proposed AZPDES permit. If the Biolac is expanded to produce class A+ effluent, the phosphorus could be removed biologically and chemically. Biological removal would require the addition of a fermentation basin ahead of the Biolac. Chemical phosphorus treatment would be achieved by precipitating the phosphorus using aluminum or iron salts. The chemical would be added to the wastewater stream between the aeration basins and the final clarifiers. The precipitated phosphorus would be removed from the wastewater with the settled sludge from the final clarifier.

5.10.1.4 Clarification

Wastewater would be treated to reduce suspended solids in an integral clarifier. The surface overflow rate would be about 400 gallons per square foot per day (gpsfd). Sludge would be pumped from the clarifier by airlift pumps to sludge thickening or dewatering facilities.

5.10.1.5 Filtration

Filtration would not be required for B+ effluent. For the class A+ effluent the low effluent phosphorus concentration would require the use of a phosphorus removal sand filter.

5.10.1.6 Disinfection

Following filtration, the effluent would be disinfected. Chlorine disinfection would be used for class B+ effluent. For class A+ effluent, either chlorine or ultra-violet disinfection would be provided.

5.10.1.7 Solids Processing

Digestion is not proposed for the sludge from the Biolac process. The long sludge age stabilizes the sludge sufficiently that it may be land applied after thickening when removed from the aeration basin.

Waste activated sludge would be pumped from the clarifier to a gravity belt thickener or other thickening process where solids would be thickened to approximately 6 % solids concentration.

5.10.1.8 Solids Disposal

Thickened solids would be land applied to agricultural properties or disposed of by an approved process.

5.10.2 EAAS

5.10.2.1 Headworks

Raw wastewater would be pumped to the treatment facility headworks, which would consist of a fine screen, solids dewatering/compaction and grit removal/ dewatering. Preliminary screening would remove all solids greater than 6 mm. Grit removal facilities would remove 80% of grit particles larger than 140 mesh. A septage receiving station will be added for receiving septage from septage haulers.

5.10.2.2 Biological Treatment (BOD_5 and Nitrogen)

From the headworks, the wastewater would flow to the EEAS basins where it would be treated for BOD_5 and nitrogen removal. Both nitrification and denitrification would be achieved biologically. Oxygen would be provided by either surface aerators or submerged diffusers. Table 5-5 shows preliminary design criteria for the EEAS treatment unit.

Table 5-5 - 20 year EAAS Design Criteria

Number of Treatment Trains	2
Basin Side water Depth, feet	16 to 20
Mixed Liquor Suspended Solids, mg/L	2,500 – 3,500
Solids Retention Time, days	20
Oxygen Requirement for BOD_5 Removal, lb O_2/lb_R	1.2
Oxygen Requirement for TKN Oxidized, lb O_2/lb_R	4.6
Alpha/Beta	.7/.95
Average Day, Peak Hour AOR/SOR, lb/h	600/1,500

5.10.2.3 Phosphorus removal

Phosphorus removal is not required for a B+ effluent. For the class A+ effluent option, the phosphorus would be removed biologically and chemically.

The biological removal would be accomplished in basins used for BOD_5 removal. Chemical phosphorus treatment would be achieved by precipitating the phosphorus using aluminum or iron salts. The chemical would be added to the wastewater stream between the aeration basins and the final clarifiers. The precipitated phosphorus would be removed from the wastewater with the settled sludge from the final clarifier.

5.10.2.4 Clarification

Wastewater would be clarified in circular or rectangular clarifiers. The surface overflow rate would be about 400 gallons per square foot per day (gpsfd). Sludge would be pumped from the clarifiers to aerobic sludge digestion facilities.

5.10.2.5 Filtration

Filtration would not be required for B+ effluent. For the class A+ effluent the low effluent phosphorus concentration would require the use of a phosphorus removal sand filter.

5.10.2.6 Disinfection

Following filtration, the effluent would be disinfected. Chlorine disinfection would be used for class B+ effluent and UV disinfection for class A+ effluent.

5.10.2.7 Solids Processing

Waste activated sludge would be pumped from the clarifiers to an aerobic digester, where solids are digested to a class B sludge quality as defined by EPA 503. Oxygenation would be provided by coarse bubble aeration or fixed mixers.

Dewatering - Solids would be either wet hauled to farmland or dewatered to 15% - 20% solids in a belt filter press or screw press.

5.10.2.8 Solids Disposal

Dewatered solids would be either land applied to agricultural properties or disposed of by an approved process.

5.10.3 SBR

5.10.3.1 Headworks

Raw wastewater would be pumped to the treatment facility headworks which will consist of a fine screen, screenings dewatering/compaction and grit removal. Preliminary screening devices would remove solids greater than 6 mm in size. Grit removal facilities would remove 80% of grit particles larger than 140 mesh. A septage receiving station will be added for receiving septage from septage haulers.

5.10.3.2 Biological Treatment (BOD_5 and Nitrogen)

From the headworks, the wastewater would flow to the SBR where it would be treated for BOD_5 and nitrogen removal. Both nitrification and denitrification would be achieved biologically. Table 5-6 summarizes the treatment design for the SBR.

Table 5-6 - 20 year SBR Design Criteria

Number of SBR Units	3
Basin Sidewater Depth, feet	16 - 20
Mixed Liquor Suspended Solids, mg/L	4,500
Solids Retention Time, days	15
Decant Rate, gpm	3,560
Oxygen Requirement for COD Removal, lb O_2/lb_R	1.2
Oxygen Requirement for TKN Oxidized, lb O_2/lb_R	4.6
Alpha/Beta	.7 /.95
Average Day, Peak Hour AOR/SOR, lb/h	750/1,200

5.10.3.3 Phosphorus removal

Phosphorus removal is not required for a B+ effluent. For the class A+ effluent option, the phosphorus shall be removed biologically and chemically. The biological removal would be accomplished in the same basins as the BOD_5 removal. Chemical treatment would be achieved by adding aluminum or iron salts into the equalization basin between the biological treatment and the filters prior to the tertiary filters.

5.10.3.4 Filtration

Filtration would not be required for B+ effluent. For the class A+ effluent the low effluent phosphorus concentration would require the use of a phosphorus removal sand filter.

5.10.3.5 Disinfection

The effluent would be disinfected following filtration. Chlorine disinfection would be used for class B+ effluent and UV disinfection for class A+ effluent.

5.10.3.6 Solids Processing

Digestion - Waste activated sludge would be pumped from the SBR tank to an aerobic digester, where solids are digested to a class B sludge quality as defined by EPA 503. Oxygenation would be provided by coarse bubble aeration or fixed mixers.

5.10.3.7 Dewatering

Solids would be either wet hauled to farmland or dewatered to 15% - 20% solids in a belt filter press or screw press.

5.10.3.8 Solids Disposal

Dewatered solids would be either land applied to agricultural properties or disposed in a landfill.

5.10.4 MBR

5.10.4.1 Headworks

Raw wastewater would be pumped to the treatment facility headworks which will consist of a coarse screen, a fine screen, solids dewatering/compaction and grit removal/dewatering. The fine preliminary screen must remove all solids greater than 1 mm. Grit removal facilities would remove 80% of grit particles larger than 140 mesh. A septage receiving station will be added for receiving septage from septage haulers.

5.10.4.2 Biological Treatment (BOD_5 and Nitrogen)

From the headworks the wastewater would flow to the MBR where it would be treated for BOD_5 and nitrogen removal. Both nitrification and denitrification would be achieved biologically. Table 5-7 shows preliminary design information for the MBR treatment units.

Table 5-7 – 20-year MBR Design Criteria

Number of MBR Units	4
Basin Sidewater Depth, feet	18
Mixed Liquor Suspended Solids, mg/L	10,000 -15,000
Solids Retention Time, days	20
Oxygen Requirement for BOD_5 Removal, lb O_2/lb_R	1.2
Oxygen Requirement for TKN Oxidized, lb O_2/lb_R	4.6
Alpha/Beta	.7/.95
Average Day, Peak Hour AOR/SOR, lb/h	600/1500

5.10.4.3 Phosphorus Removal

Phosphorus removal is not required for a B+ effluent. For the class A+ effluent option, the phosphorus shall be removed biologically and chemically. The biological removal would be accomplished in the same basins as the BOD_5 removal. . Chemical treatment would be achieved by adding aluminum or iron salts into the line between the biological treatment and the tertiary filters.

5.10.4.4 Filtration

The membranes provide both clarification and filtration of the wastewater. Effluent from the MBR would flow into an equalization basin, from where it would flow to disinfection facilities.

5.10.4.5 Disinfection

Following filtration, the effluent would be disinfected. Chlorine disinfection would be used for class B+ effluent and UV disinfection for class A+ effluent.

5.10.4.6 Solids Processing

Digestion - Waste activated sludge would be pumped from the MBR tank to an aerobic digester, where solids are digested to a class B sludge quality as defined by EPA 503. Oxygenation would be provided by coarse bubble aeration or fixed mixers.

5.10.4.7 Dewatering

Solids would be either wet hauled to farmland or dewatered to 15% - 20% solids in a belt filter press or screw press.

Dewatered solids would be either land applied to agricultural properties or disposed of by an approved process.

6.0 ALTERNATIVES 2 THROUGH 5 - TREATMENT PROCESS COST ESTIMATES

This section presents opinions of the cost to construct and operate the treatment processes previously discussed. Due to the planning nature of this project, cost estimates presented in this master plan are conceptual in nature. These estimates are intended to be used only for comparison and screening of alternatives.

6.1 Capital Cost

Capital costs presented in this report have a class 4 order of magnitude as defined by the Association for the Advancement of Cost Estimating (AACE). Specific details used in the estimates are contained in Appendix C - Methodology Used to Establish Treatment Plant Cost Estimates.

The accuracy of costs at this planning stage of the project development are normally expected to be +50% to -30% of the actual construction cost because of the absence of a detailed design to base costs on. Nevertheless, in spite of the low accuracy the cost estimates are still valuable for planning and illustration purposes and are reliable for comparing alternatives.

Table 6-1 contains Preliminary Estimate of Probable Construction Costs for the selected treatment alternatives, based upon preliminary designs for the selected processes. This includes the basins and treatment units sizing for both A+ and B+ effluent for the USFS site.

Alternative 1 and Alternative 2a do not include A+ effluent costs or grit removal. Alternative 1 cannot be modified to produce A+ effluent economically. Alternative 2a can be upgraded to produce A+ effluent for the same estimated differential cost for B+ to A+ effluent shown for Alternative 2b ($14M).

Table 6-1 – Preliminary Estimate of Probable Construction Costs – Show Low WRF (10^6 dollars)[1]

Alternative →	1	2a		2b		3		4		5	
Process	**Upgrade Existing Lagoons 2.46 mgd[2]**	**Existing Site Biolac Class B+ 1.75 mgd[3]**	**Existing Site Biolac Class B+ 2.53 mgd**	**Biolac Class A+**	**Biolac Class B+**	**EAAS Class A+**	**EAAS Class B+**	**SBR Class A+**	**SBR Class B+**	**MBR Class A+**	**MBR Class B+**
Raw Pumping	$0.0	$0.0	$0.0	$0.5	$0.5	$0.5	$0.5	$0.5	$0.5	$0.5	$0.5
Prelim Ttmt	$0.3	$0.3	$0.3	$1.9	$1.9	$1.9	$1.9	$1.9	$1.9	$2.1	$2.1
Biological treatment	$2.2	$3.2	$5.0	$6.8	$3.1	$6.7	$6.4	$6.6	$6.3	$10.4	$10.4
Secondary Clarification	$0.0	$0.7	$1.0	$1.4	$0.7	$1.6	$1.6	$0.0	$0.0	$0.0	$0.0
ChemPhos Removal	$0.0	$0.0	$0.0	$3.1	$0.0	$3.2	$0.0	$3.2	$0.0	$3.2	$0.0
Tertiary Filtration	$0.0	$0.0	$0.0	$1.5	$0.0	$1.5	$0.0	$2.5	$0.0	$0.0	$0.0
Disinfection (chlorine)	$0.0	$0.0	$0.3	$0.6	$0.6	$0.6	$0.6	$0.6	$0.6	$0.6	$0.6
Disinfection UV	$0.0	$0.0	$0.0	$1.2	$0.0	$1.2	$0.0	$1.2	$0.0	$1.2	$0.0
Aerobic Digestion	$0.0	$0.0	$0.0	$1.9	$1.9	$1.9	$1.9	$1.9	$1.9	$1.9	$1.9
Solids Dewatering[4]	$0.0	$0.7	$1.1	$3.5	$3.5	$3.5	$3.5	$3.5	$3.5	$3.5	$3.5
Standby Power	$0.0	$0.2	$0.2	$0.2	$0.2	$0.2	$0.2	$0.2	$0.2	$0.2	$0.2
Reclaimed Water Storage Tank	$0.0	$0.0	$0.0	$0.5	$0.5	$0.5	$0.5	$0.5	$0.5	$0.5	$0.5
Effluent Pumping	$0.2	$0.2	$0.2	$0.4	$0.4	$0.4	$0.4	$0.4	$0.4	$0.4	$0.4
Yard piping	$0.0	$0.5	$0.8	$1.6	$1.6	$2.5	$1.9	$2.5	$1.7	$2.6	$2.3
Odor Control	$0.0	$0.0	$0.0	$1.0	$1.0	$1.5	$1.5	$1.3	$1.3	$1.1	$1.1
Subtotal Treatment	**$2.6**	**$5.7**	**$8.8**	**$26.1**	**$15.9**	**$27.7**	**$20.9**	**$26.8**	**$18.8**	**$28.2**	**$23.5**
Administration Building/ shop	$0.4	$0.4	$0.4	$0.4	$0.4	$0.4	$0.4	$0.4	$0.4	$0.4	$0.4
Site Improvements	$0.0	$0.2	$0.4	$0.9	$1.1	$1.1	$0.9	$1.1	$0.8	$1.1	$1.0
Subtotal	$3.0	$6.3	$9.6	$27.4	$17.4	$29.2	$22.2	$28.3	$20.0	$29.7	$24.9
Contractor OH&P (15%)	$0.4	$0.9	$1.4	$4.1	$2.6	$4.3	$3.3	$4.2	$3.0	$4.5	$3.7
Contingencies (25%)	$0.7	$1.6	$2.4	$6.8	$4.3	$7.2	$5.6	$7.1	$5.0	$7.4	$6.2
Total Estimated Construction Cost	**$4.1**	**$8.8**	**$13.4**	**$38.3**	**$24.3**	**$40.7**	**$31.1**	**$39.6**	**$28.0**	**$41.6**	**$34.8**

[1] Estimated costs based on an average daily flow of 2.53 mgd, except as noted.
[2] The 2.46 MGD flow was used to match the existing APP and AZPDES permit flows.
[3] In this application, the Biolac can only be used to treat flows up to 75% greater than current flows (1 mgd current).
[4] Alternative 2a includes thickening only. Alternatives 3, 4 and 5 include dewatering.

6.2 Operation and Maintenance Cost

The anticipated annual operation and maintenance cost of the plants being considered is important when choosing a treatment process. Operation and maintenance costs include labor, power, chemicals and replacement of consumable materials such as ultraviolet light lamps and MBR membranes.

The following information was used in preparing the O&M cost estimates.

- Staffing time estimates are based upon:
  - EPA publication entitled "Estimating Costs for Conventional Wastewater Treatment Facilities".
  - Estimates from manufacturers for specific treatment processes not included in the EPA publication.
  - Wood/Patel's experience from similar processes.
- Raw labor rates of $50/hr. is used for senior management, $30/hr. for certified operators, laboratory technicians and maintenance technicians, and $20/hr. for site maintenance staff.
- Power cost of $0.10/kilowatt-hour (includes usage, demand and transmission charges).

Table 6-2 presents the planning level estimates of the cost to operate and maintain the wastewater treatment plants treating the 20 year design flows and loads. Estimated costs are based upon the design flows and current cost figures. Cost escalation is not included in the estimates.

Table 6-2 – Estimated O&M Cost – Show Low WRF (10^3 dollars)[1]

Alternative →	**1**	**2a**		**2b**		**3**		**4**		**5**	
Process	**Upgrade Existing Lagoons 2.46 mgd**	**Existing Site Biolac 1.75 mgd**	**Existing Site Biolac 2.53 mgd**	**Biolac Class A+**	**Biolac Class B+**	**EAAS Class A+**	**EAAS Class B+**	**SBR Class A+**	**SBR Class B+**	**MBR Class A+**	**MBR Class B+**
Raw Pumping	0	0	0	$60	$60	$60	$60	$60	$60	$60	$60
Prelim treatment	$10	$10	$30	$50	$50	$50	$50	$50	$50	$60	$60
Biological treatment	$310	$140	$240	$270	$240	$320	$290	$320	$290	$450	$450
Secondary Clarification	0	$20	$20	$20	$20	$20	$20	0	0	0	0
ChemPhos Removal	0	0	0	$70	0	$70	0	$70	0	$70	0
Tertiary Filtration	0	0	0	$50	0	$50	0	$50	0	$50	0
Disinfection (chlorine)	$40	$30	$60	$30	$60	$30	$60	$30	$60	$30	$60
Disinfection (UV)	0	0	0	$80	0	$80	0	$80	0	$80	0
Aerobic Digestion	0	0	0	$100	$100	$100	$100	$100	$100	$100	$100
Solids Dewatering/disposal	0	$30	$50	$100	$100	$100	$100	$100	$100	$100	$100
Odor Control	0	0	0	$50	$50	$100	$100	$100	$100	$100	$100
Effluent Pumping	$40	$30	$60	$60	$60	$60	$60	$60	$60	$60	$60
Administration	$40	$40	$60	$80	$60	$80	$60	$80	$60	$80	$60
I&C Maintenance	$5	$10	$10	$20	$20	$40	$20	$40	$20	$40	$20
Site Maintenance	$10	$10	$10	$30	$30	$ 30	$30	$30	$30	$30	$30
Laboratory	$10	$30	$30	$40	$30	$40	$30	$40	$30	$40	$30
General Utilities	$5	$10	$10	$20	$20	$30	$30	$30	$30	$30	$30
Misc.(PR, seminars, memberships, etc.)	$10	$20	$20	$20	$20	$20	$20	$20	$20	$20	$20
Total Estimated Annual O&M Cost	**$480**	**$380**	**$600**	**$1,150**	**$920**	**$1,280**	**$1,040**	**$1,260**	**$ 1,010**	**$1,400**	**$ 1,180**

[1] See foot notes 1, 2, and 3 in Table 6-1.

6.3 Life Cycle Costs

Life cycle costs are long term costs that include both the initial capital outlay as well as annual O&M costs. Life cycle costs in this report are based upon a real discount rate of 3% per year for the capital improvements.

Table 6-3 presents the estimated life cycle cost comparison for the five alternatives. The costs reflect the annual cost over the plant life (20 years). The Phase 1 Biolac alternative would be designed for a lower flow than the other alternatives, requiring that it be expanded in approximately 10 years. Thus the discount rate for the Phase 1 Biolac capital cost is 0.11723 compared to 0.0672 for the 20-year life of Phase 2 used for the other alternatives.

As shown in Table 6-3, Alternative 1, upgrade the existing lagoons, has the lowest annual cost. Although Alternative 1 represents the lowest costs if the alternatives analyzed, it is not the recommended alternative as will be explained later.

Table 6-3 shows that Alternative 2a, the Biolac at the existing site, has the second lowest annual life cycle cost and approximately 70% of the cost of the SBR on a cost per gallon basis. Alternative 2a discussed herein will not produce a class A+ effluent.

Alternative 2b, Biolac at the USFS site, will have the lowest annual life cycle cost for a 20 year plant capable of producing both a class A+ and class B+ effluent at the USFS site. The EAAS plant is relatively close in cost to the SBR; and considering the level of accuracy of these estimates, the two processes may be considered equivalent from a cost standpoint.

The MBR process has the highest annual cost than the other alternatives for the class B+ effluent. By its nature, the MBR would produce a higher quality effluent than B+, even though discharge into the wetlands does not require it. For the A+ effluent, the life cycle cost of the MBR is higher than the other alternatives.

Table 6-4 presents a summary of estimated costs for all alternatives along with flows treated.

Table 6-3 – Estimated Annual Life Cycle Cost – Show Low WRF (10^6 dollars)[1]

Alternative →	**1**	**2a**		**2b**		**3**		**4**		**5**	
	Upgrade Existing Lagoons 2.46 mgd	**Biolac Existing Site 1.75 mgd**	**Biolac Existing Site 2.53 mgd**	**Biolac Class A+**	**Biolac Class B+**	**EAAS Class A+**	**EAAS Class B+**	**SBR Class A+**	**SBR Class B+**	**MBR Class A+**	**MBR Class B+**
Annual Capital Cost	$0.27	$1.03	$0.90	$2.57	$1.63	$2.71	$2.09	$2.66	$1.88	$2.79	$2.34
Total Estimated Annual O&M	$0.48	$0.38	$0.60	$1.15	$0.92	$1.28	$1.04	$1.26	$1.01	$1.40	$ 1.18
Total Annual Cost	**$0.75**	**$1.41**	**$1.50**	**$3.72**	**$2.55**	**$3.99**	**$3.13**	**$3.92**	**$2.89**	**$4.19**	**$3.52**

[1] See foot notes 1,2,and 3 in Table 6-1.

Table 6-4 – Summary of Estimated Costs – Show Low WRF

Alternative →	**1**	**2a**		**2b**	**3**	**4**	**5**
ITEM	**Upgrade Existing Lagoons**	**Existing Site Biolac Class B+**		**USFS Site Biolac Class B+**	**EAAS Class B+**	**SBR Class B+**	**MBR Class B+**
Flow, MGD	2.46	1.75	2.53	2.53	2.53	2.53	2.53
Capital Cost – $10^6$$	$4.1	$8.8	$13.4	$24.3	$31.1	$28.0	$34.8
O&M - $10^6$$	$0.5	$0.4	$0.6	$0.9	$1.0	$1.0	$1.2
Life Cycle - $10^6$$[1]	**$0.8**	**$1.4**	**$1.5**	**$2.6**	**$3.1**	**$2.9**	**$3.5**

[1] Life cycle costs are long term costs that include both the initial capital outlay as well as annual O&M costs. Life cycle costs in this report are based upon a real discount rate of 3% per year for the capital improvements.

7.0 ALTERNATIVES 1 THROUGH 5 - NON-ECONOMIC EVALUATION

Numerous factors other than economics influence the decision of which alternative to implement. Some of these factors are as follows:

- Public Acceptability
- Land area required
- Future ADEQ permit requirements
- Future 208 plan requirements
- Ability of the receiving waters to receive the future flow
- Ability to alternate between Class B+ and Class A+ effluent.
- Comparative energy consumption
- Ability to transition from the existing treatment plant to the new treatment plant.
- Ability to construct the improvements in phases.
- Non-economic impact of residuals disposal

Each of these factors is addressed below.

7.1 Matrix Comparison

Table 7-1 contains a comparative matrix, which provides a subjective rating of the benefits/detriments of each process. The following parameters were considered as non-economic factors.

<u>Public Acceptability</u>

All three alternatives will be acceptable to the public because they would be constructed on a remote site which will allow the existing site to be utilized for other purposes. Both the class B+ treatment and A+ treatment are acceptable for their own reasons. The existing wetlands are well accepted by the public, so continued use of the wetlands is desirable.

Similarly, some desire exists within Show Low that reclaimed water be available for irrigation and recreation purposes. Class A+ effluent will satisfy these desires. Thus, treatment to both a B+ level and A+ level will be accepted by the public.

Land Area

All three processes require several buildings and/or structures. The EAAS requires the most structures at 12, and the MBR requires the least at 10. The EAAS will require clarifiers and sludge pumping, which an SBR will not require. Thus, the EAAS will require the most land area and the MBR the least area.

Future ADEQ Permit Requirements

The existing ADEQ permit does not contain nitrogen limitations for the class B+ effluent; however, it appears that future permits WILL place a restriction on nitrogen. All three alternatives can be designed and operated to achieve Class A+ effluent which includes nitrogen removal. Therefore none of the processes contain an advantage over another in this regard.

Future 208 Requirements

The Northern Arizona Council of Governments (NACOG) will determine the degree to which changes to the 208 plan will be required for the Show Low wastewater improvements. It is not envisioned that any of the alternatives will have an advantage, or disadvantage over another in this regard.

Receiving Water Capacity - Wetlands

Section 8 of the "***City of Show Low Phase 2 Analysis of Wastewater Treatment Plant***", dated October 26, 2009, states the following,

"The Show Low WWTP constructed wetlands are used to dispose of the wastewater effluent following flow through the polishing wetlands. The constructed wetlands have been operational for the disposal of treated effluent since 1970 and currently consist of 250 surface acres with a storage capacity of 957 acre-feet.

The wetlands have a calculated rate of net evaporation of 3.25 feet per year. This accounts for 71 percent of the water loss in the wetlands. The remaining 29 percent is attributed to seepage.

The wetlands are nearing capacity at current average daily flow of 1.02 MGD. Their capacity could be increased from 372 million gallons per year to 500 million gallons per year with water management as follows:

- *Allowing wetlands to spill into natural drainages, or*
- *Creating additional wetland or riparian areas.*

Each expansion may be taken on an as-needed basis by the City to meet the projected increases in flows to the WWTP. Wetlands capacity expansions to approximately 500 million gallons per year is equivalent to an average daily inflow of 1.37 MGD.

Any improvements or expansion of the existing wetlands would require the approvals from the USFS through the NEPA process and ADEQ review and approval."

This analysis from the Phase 2 Report indicates that existing wetland capacity to receive Class B+ effluent is less than the future design flow. Thus, additional efforts will be required to expand the wetland disposal capacity.

In the event that the wetland capacity is not upgraded, the required level of treatment for a portion of the wastewater must be Class A+. All three treatment alternatives can be constructed to initially produce a Class B+ effluent, and upgraded to A+ at a later date. Thus, there is no relative benefit or detriment for any of the alternatives in this regard.

Surface Water

Table 5-1 shows that A+ water quality is required for discharge into surface waters. All three alternatives can be designed to produce A+ effluent, so no relative benefit or detriment exists for any alternative in this regard.

Alternate Treatment between A+ and B+ Effluent

Table 6-3 shows that it is between 20% and 40% less costly to produce B+ effluent than A+ effluent. It is therefore prudent to produce B+ effluent whenever possible.

The difference in O&M cost for Class A+ and B+ is due to reduced chemicals for phosphorus removal, reduced power for UV disinfection, elimination of the filtration phase and reduced power for the biological nutrient removal process. The operation of all three processes can be adjusted to switch back and forth between Class A+ and B+ treatment; however, none of the processes can be modified rapidly. The EAAS will require the most effort to alternate between Class B+ and A+ because treatment units would be taken out of service. The SBR would require less effort because much of the treatment modifications would be operational within existing basins. The MBR would be easiest to alternate since no processes would be taken out of service.

Comparative Energy Requirements

The MBR process consumes more energy than the Biolac, SBR or EAAS processes. The EAAS consumes more energy than the Biolac and the SBR, with the SBR being the most efficient. When producing A+ effluent, the Biolac, EAAS and SBR must utilize filtration. Energy requirements for filtration largely eliminate the advantage the Biolac, SBR and EAAS processes have over the MBR.

Ability to Transition from Existing Treatment Plant to Future Treatment Plant

The new treatment plant will be constructed on a site remote from the existing plant. There will be no benefit to any particular alternative regarding transition from the existing plant to the new plant.

Ability to construct the improvements in phases

The existing peak month flow is approximately 1.0 MGD while the 2031 design year flow is projected to be 2.53 MGD. For cost and efficiency purposes, it is recommended the treatment plant be constructed in phases to match the growth. All three alternatives can be designed for phased construction. The EAAS would be the most difficult to construct in phases due to the need for aeration, clarification and pumping. The SBR can readily be constructed in phases due to the single tank treatment. The MBR can also be readily constructed in phases because the future membrane units can be easily added.

Non-economic impact of residuals disposal

All three alternatives propose to produce Class B biosolids. Biosolids meeting this classification may be either applied to agricultural land or to landfills. If land applied, site restrictions must be met. Since all three alternatives would utilize the same biosolids treatment process no alternative has advantage over another.

7.2 Ranking of Non-economic Parameters

Table 7-1 presents a matrix of the Non-Economic Evaluation of the treatment Alternatives. A rating of 1 represents lowest relative acceptability. A rating of 5 represents highest acceptability. A rating of 3 represents no relative advantage over the other alternatives.

All alternatives are relatively equal for the non-economic benefits, with the Biolac at the existing site being the most desirable and the EAAS the least desirable.

Table 7-1 – Rating of Non–Economic Evaluation of Alternatives[1]

	Upgrade Lagoons	**Biolac at Exist. Site**	**Biolac at USFS**	**EAAS**	**SBR**	**MBR**
Public Acceptability	4	4	3	3	3	3
Land Area	5	5	3	2	3	4
ADEQ Permit Requirements	3	3	3	3	3	3
208 Requirements	3	3	3	3	3	3
Receiving water capacity	3	3	3	3	3	3
Alternate treatment between A+, B+	1	2	3	2	3	4
Energy requirements	3	3	3	3	4	2
Transition of old to new plant	5	4	3	3	3	3
Phase construction	3	4	3	3	4	4
Residual disposal	1	2	3	3	3	2
Total points	31	33	30	28	32	31

[1]Rating of 1 is least desirable and 5 the most desirable. Rating of 3 is neutral.

8.0 RECOMMENDED WASTEWATER TREATMENT PROCESS

Table 6-3 shows that the Alternative 2a, Biolac at the existing WWTP site, has the lowest estimated capital and O&M costs of the alternatives evaluated. Table 7-1 shows that Alternative 2a also has the highest rating for the Non-Economic parameters considered. Therefore, it is recommended that the City pursue implementation of Alternative 2a, Biolac at the existing WWTP site, for upgrading the wastewater treatment plant. Figure 5-15 presents a preliminary layout of a Biolac treatment plant constructed on the existing site. Figures 5-16 and 5-17 present the proposed locations of the phase 1 and 2 Biolac facilities within the existing wastewater treatment plant site.

The Arizona Administrative Code, (R18-9-B201) requires that the owner of a sewage treatment facility that is a new facility or undergoing a major modification provide setbacks established by code. Setbacks are measured from the treatment and disposal components within the sewage treatment facility to the nearest property line of an adjacent dwelling, workplace, or private property. If an owner cannot meet a setback for a facility undergoing a major modification that incorporates full noise, odor, and aesthetic controls, the owner shall not further encroach into setback distances existing before the major modification except the owner may decrease setbacks if setback waivers are obtained from affected property owners in which the property owner acknowledges awareness of the established setbacks, basic design of the sewage treatment facility, and the potential for noise and odor. The City currently holds setback waivers from adjacent property owners. However, these waivers may require updating prior to ADEQ approving the proposed facility upgrades. Figure 5-18 shows the location of the setback distances from the proposed facility and their impact on the surrounding properties.

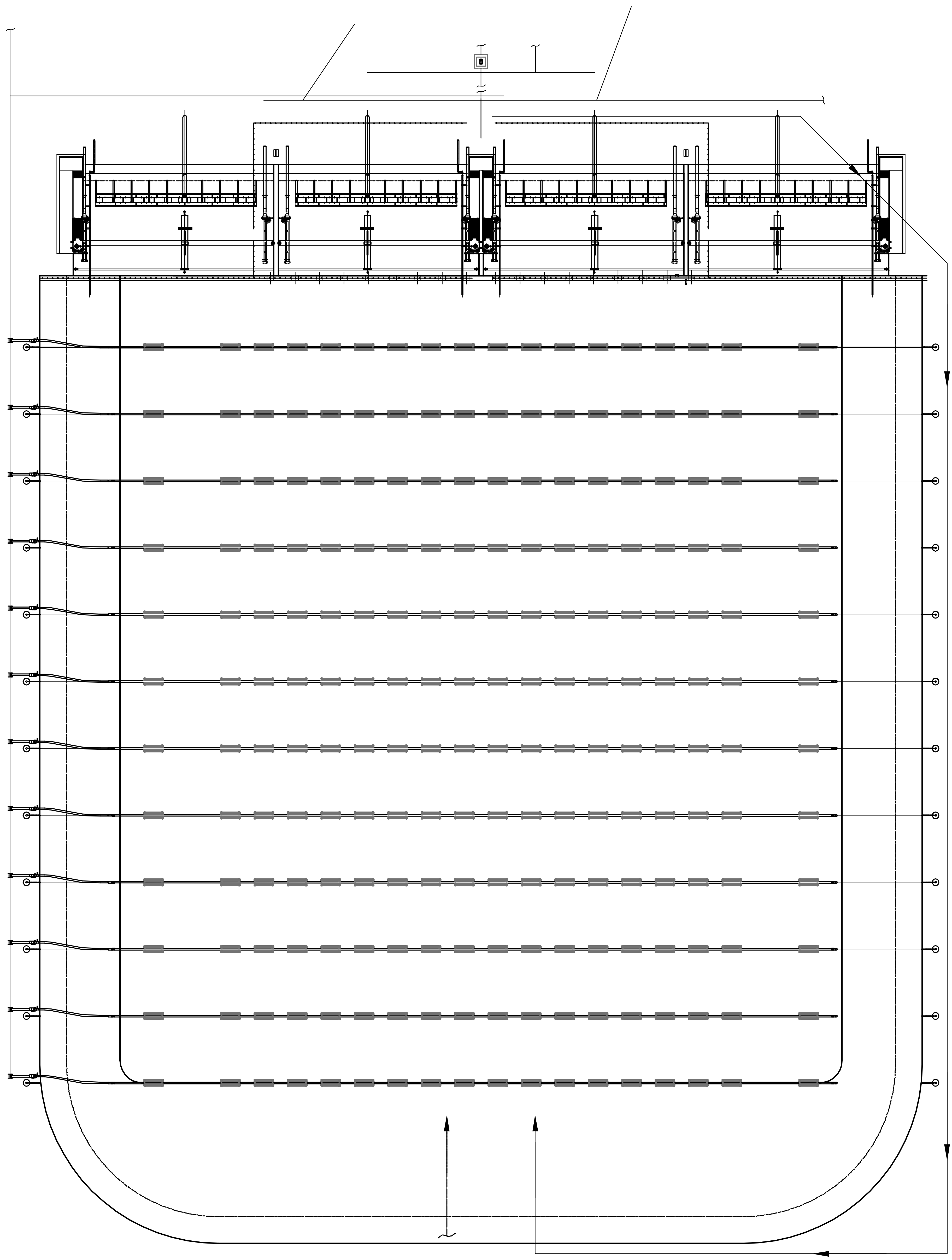

FIGURE 5-15

BIOLAC SYSTEM BASIN

CITY OF SHOW LOW
WASTEWATER MASTER PLAN

WOOD/PATEL
MISSION: CLIENT SERVICE™
PHOENIX • MESA • TUCSON

2051 W. Northern Ave.
Phoenix, AZ 85021
(602) 335-8500

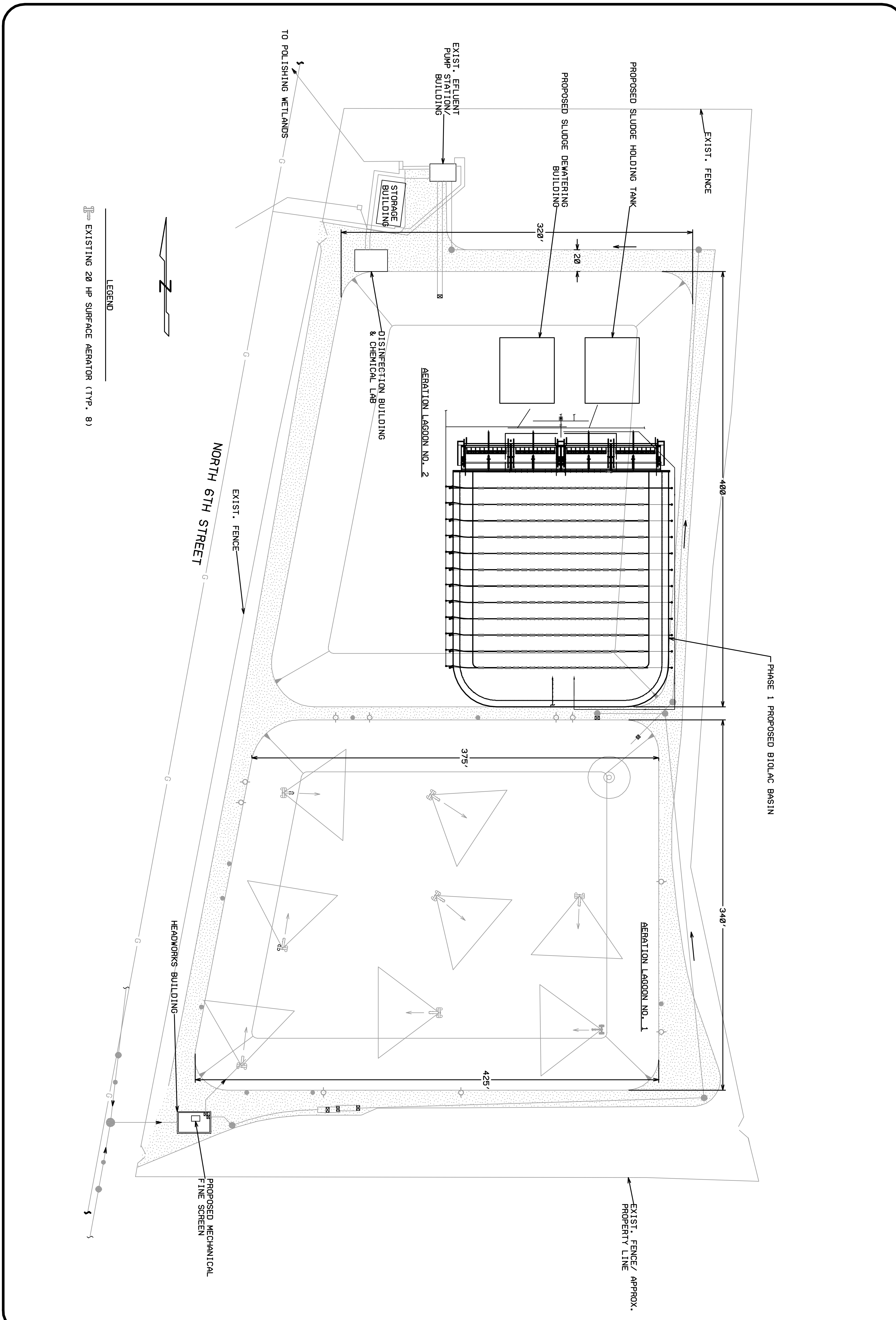

FIGURE 5–16

PHASE 1 BIOLAC SITE PLAN

CITY OF SHOW LOW
WASTEWATER MASTER PLAN

WOOD/PATEL
MISSION: CLIENT SERVICE™
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2051 W. Northern Ave.
Phoenix, AZ 85021
(602) 335-8500

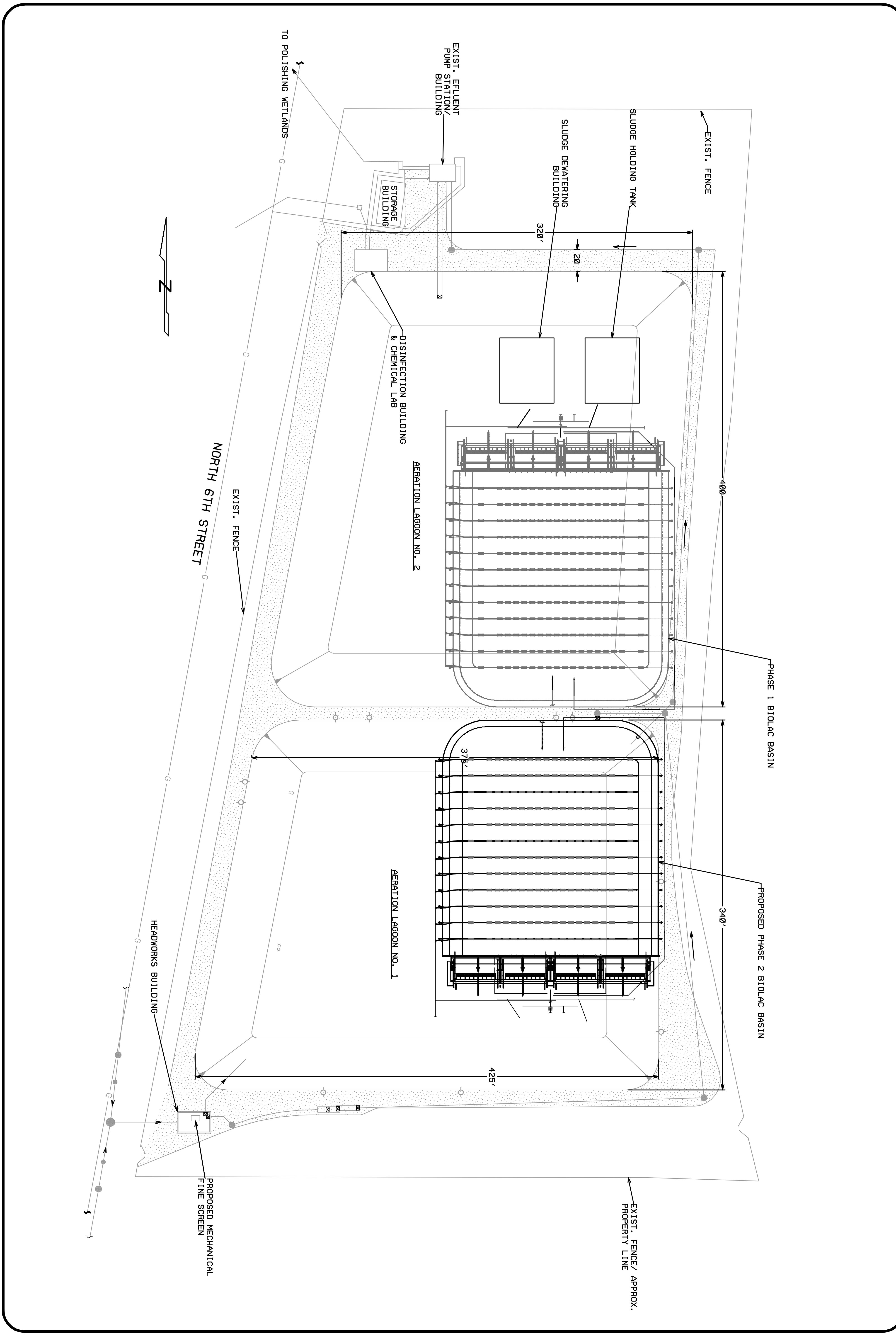

FIGURE 5–17

PHASE 2 BIOLAC SITE PLAN

CITY OF SHOW LOW
WASTEWATER MASTER PLAN

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APN	PROPERTY OWENER	ACRE	TYPE
210-02-018	MOUNTAIN LANE HOLDINGS LLC	37.71	MIXED USE
210-02-016-B	PERKINS, BEN L JR & THORA TRUST	10.46	MIXED USE
210-02-016-A	PERKINS, BEN L JR & THORA TRUST	0.39	COMMERCIAL
210-02-009-B	STOCK, GREGORY & MONICA	30.5	AGRICULTURAL
210-02-002-Q	STOCK, GREGORY & MONICA CPRS	1.83	VACANT
210-02-002-R	TROUTMAN, RON L & ITKA K CPRS	2.17	RESIDENTIAL
210-02-002-T	STOCK, GREG K & MONICA CPRS	8	VACANT

FIGURE 5-18

SETBACK EXHIBIT

CITY OF SHOW LOW
WASTEWATER MASTER PLAN

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9.0 PERMITTING/REGULATORY REQUIREMENTS

The construction of the proposed Show Low WWTP would require several reviews and approvals and the issuance of several permits from a number of Federal, State, and local regulatory agencies. Future permits and approvals required are addressed in the following paragraphs and are summarized in Table 9-1.

9.1 208 Plan Amendment

An amendment to the Areawide Water Quality Management Plan in accordance with Section 208 of the Clean Water Act must be obtained prior to obtaining any reviews or approvals of any other facets of the project. The Northern Arizona Association of Governments (NACOG) administers this program for the Show Low area for USEPA. The 208 Plan Amendment must be approved before the Arizona Department of Environmental Quality (ADEQ) will initiate a discharge permit (AZPDES) or Aquifer Protection Permit (APP) review process. The City would be required to provide a letter to ADEQ stating that the proposed WRF will be City owned. Additionally, letters from potentially impacted Native American Tribe must be sent to ADEQ stating their approval or concerns with the 208 Plan Amendment. ADEQ would inform the State Historic Preservation Office during the 208 Plan Amendment process. It is expected that the 208 Plan Amendment approval may take nine months to one year to complete after the proposed amendment is submitted.

9.2 Aquifer Protection Permit (APP)

An APP must be obtained from ADEQ in order for the City to go forward with construction of the plant. The proposed facility would have to be designed to meet ADEQ's requirements for the Best Available Demonstrated Control Technology (BADCT). BADCT requirements include:

- Compliance with setback requirements
- Conformance with the 208 Plan
- Define discharge impact area which can include a 20-year and 40-year hydrologic particle tracking analysis
- Compliance with treatment performance standards

- BOD_5: <30 mg/L
- TSS: <30 mg/L
- Total N: $\leq$ 10 mg/L (5 month rolling geometric mean)
- Maximum seepage rate through earthen containment structures: $\leq$ 550 gpd/acre
- Fecal coliforms: 200 cfu/100 ml
- Meet numeric aquifer water quality standards for all other pollutants
- Minimize the formation of trihalomethanes (THM's) with chlorination/ dechlorination and ultraviolet disinfection.

- Submittal of a design report that includes:
  - Wastewater characterization
  - Unit treatment process descriptions
  - Description of planned operations and maintenance
  - Description of construction management controls
  - System startup plan
  - Site diagram showing setback compliance
  - Design plans
  - Demonstration of technical and financial capability
  - Identify all potential reclaimed water users
  - Submittals must be in triplicate

Scheduling of a pre-application meeting for the project is recommended by ADEQ. ADEQ will charge a fee for all reviews, hearings and meetings. Public notices and public hearings will be required prior to issuing the APP.

9.3 Discharge Permit

This National Pollutant Discharge Elimination System (NPDES/AZPDES) permit would be required for the discharge of effluent to any Waters of United States. There can be no objection from effected parties, including Native American Tribes and others, downstream of the point of discharge.

9.4 Reuse Agreement

A reuse agreement would need to be in place before reclaimed water can be delivered. The agreement would need to be submitted to ADEQ for review and approval. ADEQ's Class A+ reclaimed water standards include:

- Secondary treatment
- Filtration
- Chemical feed facility to add coagulants or polymers to ensure turbidity removal
- Disinfection
- Meet water quality criteria
    - Turbidity: <2 NTU (24 hour average) and never >5NTU
    - Fecal coliforms: 0 cfu/100 ml single sample maximum
    - Total N: <10 mg/L (5 sample geometric mean)

9.5 Flood Plain Use Permit

If construction is within the 100 year flood plain, the facilities would have to be placed at either a minimum grade of one foot above the 100-year high water elevation or flood protection measures, such as berming around the facility, be taken such to protect it from flooding.

9.6 Well Drilling Permit

Drilling permits would be required for any groundwater monitoring wells constructed for the purpose of determining compliance at the WRF. These permits would need to be obtained from the Arizona Department of Water Resources (ADWR).

9.7 Storm water Pollution Prevention Plan

A Storm water Pollution Prevention Plan (SWPPP) would need to be prepared and submitted for approval prior to initiating any grading activities. This permit is required to ensure that appropriate measures are taken to prevent erosion and sedimentation of waterways. This program is administered by the Navaho County Flood Control Department for ADEQ.

9.8 Sludge Disposal Permit

Disposal of sludge from the proposed WRF would need to be compliant with US Code of Federal Regulations 40 CFR Part 503. If sludge from the proposed Show Low WWTP is to be processed and disposed of at another facility then that facility's permit may have to be amended.

9.9 Air Quality Permit

In order to operate a natural gas fueled engine generator at the proposed WRF, an air quality permit may be required.

9.10 Historic and Prehistoric Artifacts

The State Historic Preservation Office (SHPO) would need to be contacted to verify the existence of any known sites in the project area of historical significance. In addition, if during the course of excavation activities any artifacts are uncovered, work in that area must be halted and SHPO contacted to determine the extent and significance of the findings.

9.11 Endangered and Threatened Species

Work may not be conducted on a site that is the habitat of known threatened or endangered species with US Fish and Wildlife Service. The project must be approved by the Arizona Game and Fish Department.

Table 9-1 - Future Permit Requirements Show Low Wastewater Master Plan

Permit	**Regulatory Agency**
208 Plan	ADEQ
Aquifer Protection Permit (APP)	Arizona Department of Environmental Quality (ADEQ)
Sludge Disposal Permit	ADEQ
AZPDES Discharge Permit	ADEQ
Air Quality Permit	Navaho County Environmental Services Department (NCESD)
Grading Permit	City of Show Low
Architectural Approval	City of Show Low
Building Permit	City of Show Low
Endangered Species	US Fish and Wildlife Service.
Historic Preservation	SHPO
Well Permit	ADWR

APPENDIX A

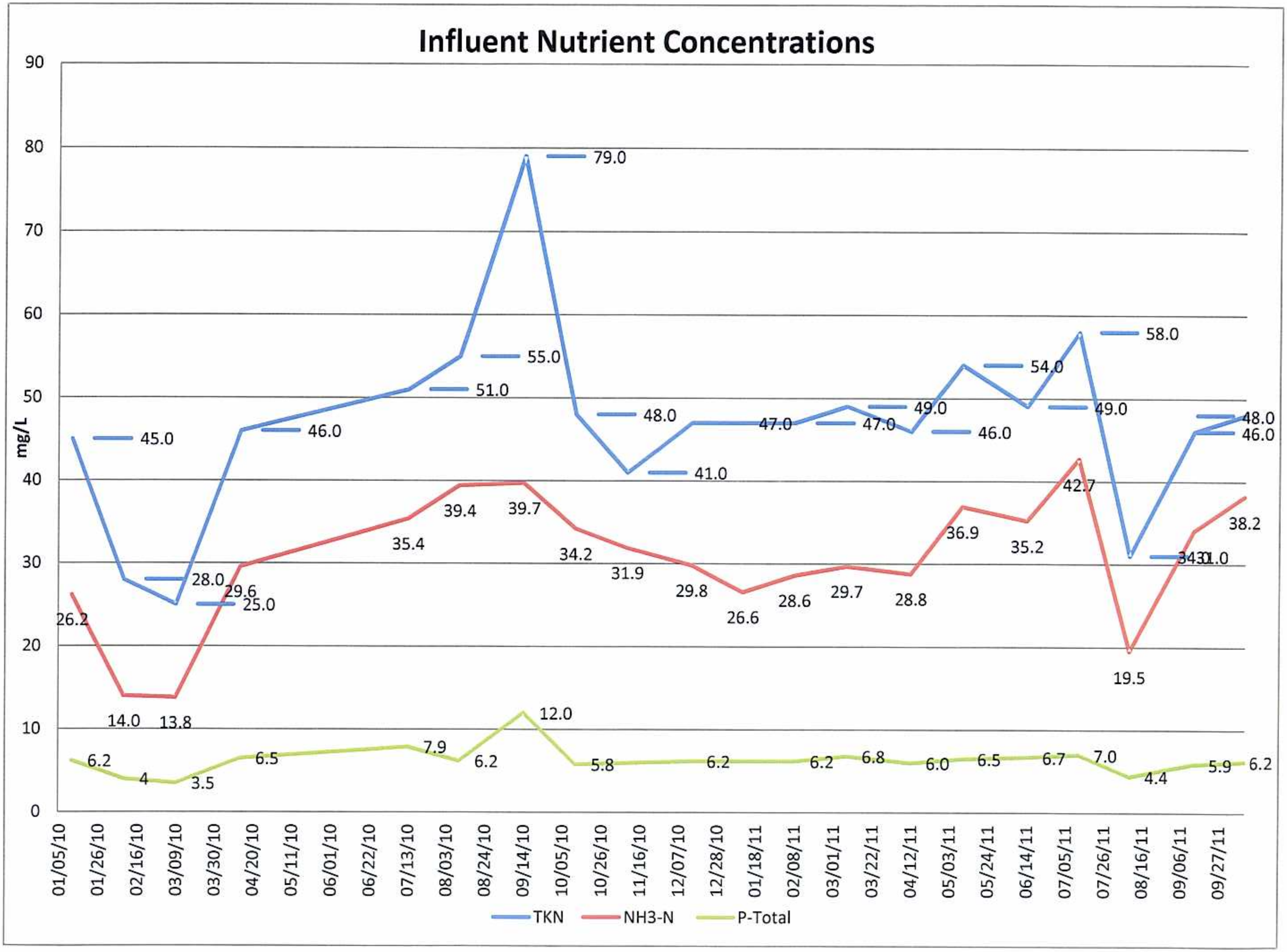
Nitrogen Limits

Determination of Chronic Total Ammonia Criteria in mg/L N										
Based on pH and Temperature at Time of Sampling (1) (2)										
	Temperature, °C									
pH	**0**	**14**	**16**	**18**	**20**	**22**	**24**	**26**	**28**	**30**
6.5	6.67	6.67	6.06	5.33	4.68	4.12	3.62	3.18	2.80	2.46
6.6	6.57	6.57	5.97	5.25	4.61	4.05	3.56	3.13	2.75	2.42
6.7	6.44	6.44	5.86	5.15	4.52	3.98	3.50	3.07	2.70	2.37
6.8	6.29	6.29	5.72	5.03	4.42	3.89	3.42	3.00	2.64	2.32
6.9	6.12	6.12	5.56	4.89	4.30	3.78	3.32	2.92	2.57	2.25
7.0	5.91	5.91	5.37	4.72	4.15	3.65	3.21	2.82	2.48	2.18
7.1	5.67	5.67	5.15	4.53	3.98	3.50	3.08	2.70	2.38	2.09
7.2	5.39	5.39	4.90	4.31	3.78	3.33	2.92	2.57	2.26	1.99
7.3	5.08	5.08	4.61	4.06	3.57	3.13	2.76	2.42	2.13	1.87
7.4	4.73	4.73	4.30	3.78	3.33	2.92	2.57	2.26	1.98	1.74
7.5	4.36	4.36	3.97	3.49	3.06	2.69	2.37	2.08	1.83	1.61
7.6	3.98	3.98	3.61	3.18	2.79	2.45	2.16	1.90	1.67	1.47
7.7	3.58	3.58	3.25	2.86	2.51	2.21	1.94	1.71	1.50	1.32
7.8	3.18	3.18	2.89	2.54	2.23	1.96	1.73	1.52	1.33	1.17
7.9	2.80	2.80	2.54	2.24	1.96	1.73	1.52	1.33	1.17	1.03
8.0	2.43	2.43	2.21	1.94	1.71	1.50	1.32	1.16	1.02	0.897
8.1	2.10	2.10	1.91	1.68	1.47	1.29	1.14	1.00	0.879	0.773
8.2	1.79	1.79	1.63	1.43	1.26	1.11	0.973	0.855	0.752	0.661
8.3	1.52	1.52	1.39	1.22	1.07	0.941	0.827	0.727	0.639	0.562
8.4	1.29	1.29	1.17	1.03	0.906	0.796	0.700	0.615	0.541	0.475
8.5	1.09	1.09	0.990	0.870	0.765	0.672	0.591	0.520	0.457	0.401
8.6	0.920	0.920	0.836	0.735	0.646	0.568	0.499	0.439	0.386	0.339
8.7	0.778	0.778	0.707	0.622	0.547	0.480	0.422	0.371	0.326	0.287
8.8	0.661	0.661	0.601	0.528	0.464	0.408	0.359	0.315	0.277	0.244
8.9	0.565	0.565	0.513	0.451	0.397	0.349	0.306	0.269	0.237	0.208
9.0	0.486	0.486	0.442	0.389	0.342	0.300	0.264	0.232	0.204	0.179
Footnotes:										
(1) pH and temperature are field measurements taken at the same time and location as the water samples destined for the laboratory analysis of ammonia.										
(2) If field measured pH and/or temperature values fall between the Chronic Total Ammonia tabular values, round field measured values according to standard scientific rounding procedures to nearest tabular value to determine the ammonia standard.										

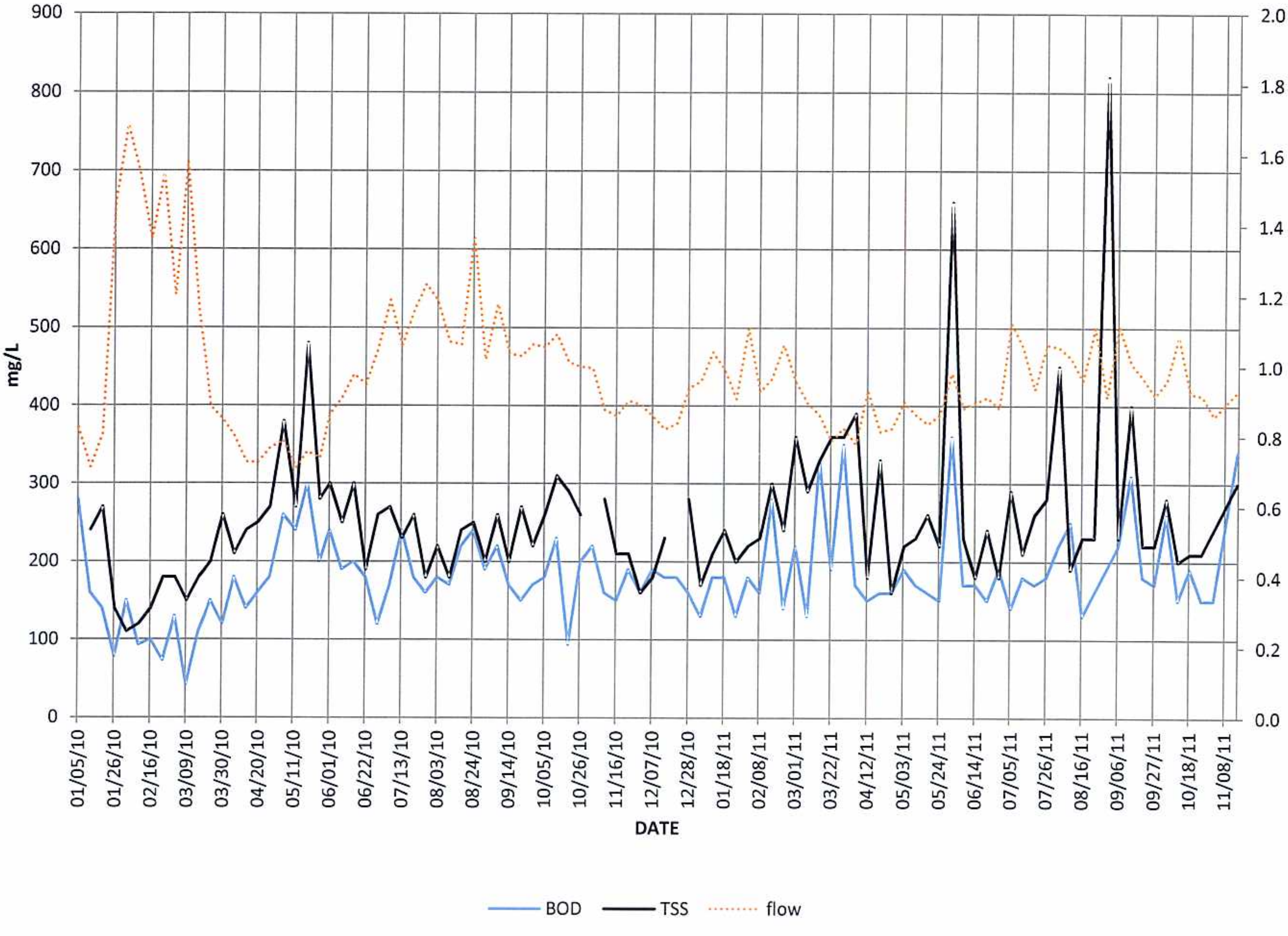
Source: Arizona Administrative Code Title 18, Chapter 11 Department of Environmental Quality Water Quality Standards

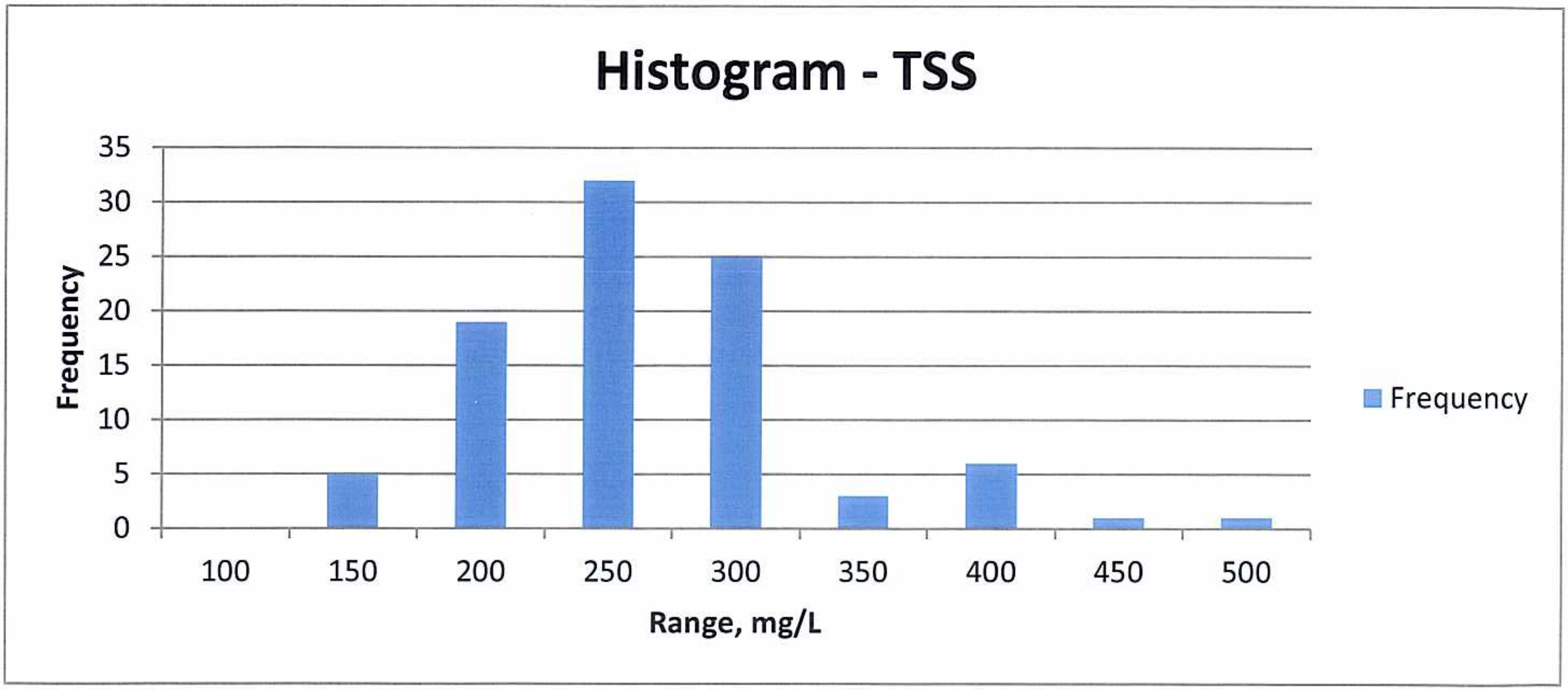
APPENDIX B

WWTP Influent Loading Data

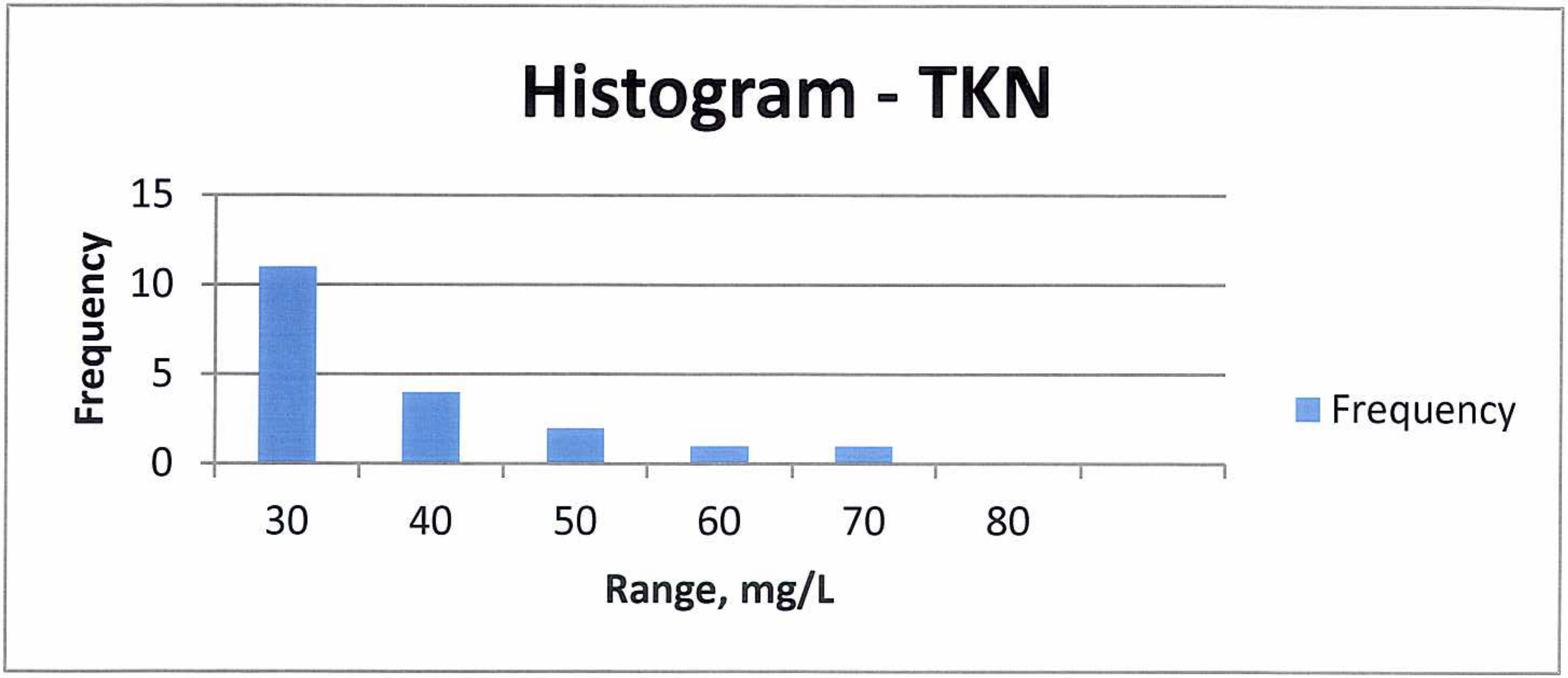

Influent Nutrient Concentrations
mg/L
90
80
70
60
50
40
30
20
10
0
01/05/10
01/26/10
02/16/10
03/09/10
03/30/10
04/20/10
05/11/10
06/01/10
06/22/10
07/13/10
08/03/10
08/24/10
09/14/10
10/05/10
10/26/10
11/16/10
12/07/10
12/28/10
01/18/11
02/08/11
03/01/11
03/22/11
04/12/11
05/03/11
05/24/11
06/14/11
07/05/11
07/26/11
08/16/11
09/06/11
09/27/11
TKN
NH3-N
P-Total
45.0
28.0
25.0
46.0
51.0
55.0
79.0
48.0
41.0
47.0
47.0
49.0
46.0
54.0
49.0
58.0
48.0
46.0
26.2
14.0
13.8
29.6
35.4
39.4
39.7
34.2
31.9
29.8
26.6
28.6
29.7
28.8
36.9
35.2
42.7
19.5
38.2
6.2
4
3.5
6.5
7.9
6.2
12.0
5.8
6.2
6.2
6.8
6.0
6.5
6.7
7.0
4.4
5.9
6.2

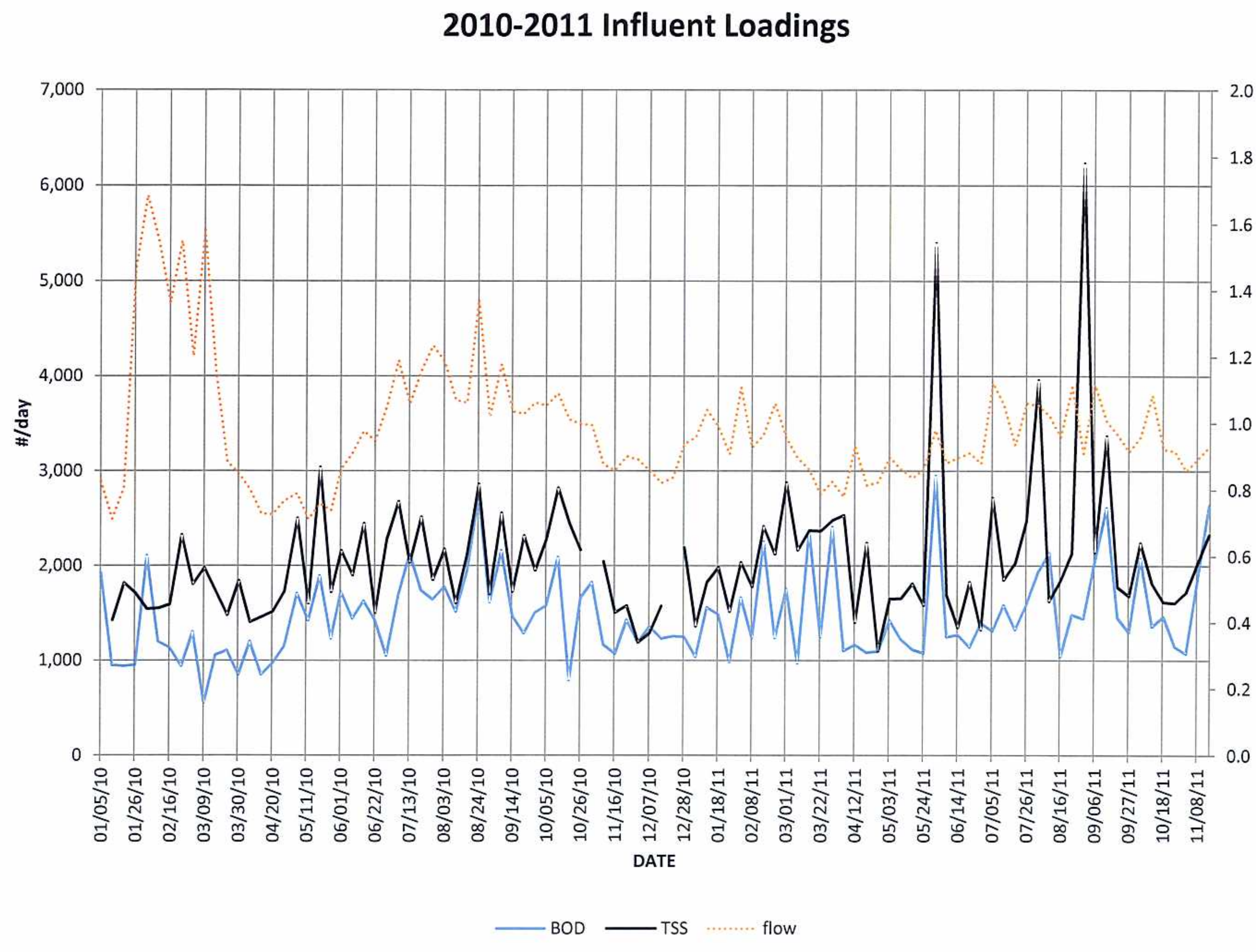
2010-2011 Influent Concentrations





Histogram - TSS
Frequency
35
30
25
20
15
10
5
0
100
150
200
250
300
350
400
450
500
Range, mg/L
Frequency


Histogram - TKN
Frequency
15
10
5
0
30
40
50
60
70
80
Range, mg/L
Frequency


2010-2011 Influent Loadings
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
#/day
2.0
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
Flow, MGD
01/05/10
01/26/10
02/16/10
03/09/10
03/30/10
04/20/10
05/11/10
06/01/10
06/22/10
07/13/10
08/03/10
08/24/10
09/14/10
10/05/10
10/26/10
11/16/10
12/07/10
12/28/10
01/18/11
02/08/11
03/01/11
03/22/11
04/12/11
05/03/11
05/24/11
06/14/11
07/05/11
07/26/11
08/16/11
09/06/11
09/27/11
10/18/11
11/08/11
DATE
BOD
TSS
flow

APPENDIX C

Methodology Used
to
Establish Treatment Plant Cost Estimates

1. The basis for the cost estimate is the equipment and structural cost for each process.

2. Equipment costs were obtained from manufacturers for the major equipment units.

3. Preliminary concrete volumes were determined for each treatment unit. The unit cost of concrete was multiplied by the calculated volumes. Unit costs used was $550/cy for slabs and $650/cy for walls and elevated slabs.

4. The cost of ancillary treatment units was determined by applying percentages of the installed treatment equipment plus structural cost. Percentages are based on averages from other projects throughout the Unites States. The following percentages were used: Excavation/backfill (10% structure cost), yard piping (10 %), equipment installation (30% equipment cost), process piping (40% of equipment cost), process electrical (15% installed equipment cost), instrumentation and controls (7 1/2% installed equipment cost), odor control (10% of the processes requiring it), process water (1% installed equipment cost), standby power ($1,000/kw).

5. The following buildings will be required: administration/laboratory/maintenance, preliminary treatment, aeration blowers, chemical feed, MBR treatment, tertiary filters, UV disinfection, solids dewatering, plant process water, electrical service/motor control. Building costs are assumed at $120/sf.

6. The following processes will not be installed within buildings, but will need to be covered to either protect against freezing, or for odor control: EAAS structures, SBR structures, clarifiers, chlorine contact tanks and aerobic digesters.

7. Site improvements will be required for the buildings and structures. The cost of each site improvement is based on a percentage of the cost of the buildings and treatment structures. The following percentages are used: Roadways (1%), drainage (1%), site electric (1%), landscaping (0.5%), natural gas (0.2%), water supply (0.2%) and fencing (1%).

8. All three plants are assumed to be constructed on a site owned by the City. No costs are included in this estimate as all three would encounter the same cost. However the total project cost should also include the cost to construct a sewer from the existing treatment plant to the new treatment plant site. Cost will include site acquisition, sewer easements and ROW, construction dewatering and force main. A new main sewage pump station may also be required.